CWRS

ON-SITE WASTEWATER DISPOSAL IN NOVA SCOTIA

G.M. CHARLES AND D.H. WALLER

FINAL REPORT

ON-SITE WASTEWATER RESEARCH PROGRAM

PHASE I

Technical University of Nova Scotia
Centre for Water Resources Studies

P.O. Box 1000
1360 Barrington Street
Halifax, Nova Scotia
Canada B3J 2X4

(902) 420-7500

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PHASE I

1987 - 1990

G. M. CHARLES and D. H. WALLER

Report to Province of Nova Scotia

from
Centre for Water Resources Studies
Technical University of Nova Scotia

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Executive Summary

With the support and cooperation of several departments of the Nova Scotian government, a three year research program was created in 1987 with the objective of imporving the effectiveness and reducing the cost of On-Site wastewater treatment systems. The project focused on two distinct areas of study: in-situ evaluation of lots; and the design assumptions and performance of the newer "C2" type systems. The program also provided an opportunity to consider experiences in the New England area which might be relevant to design and/or regulations of systems in Nova Scotia. A total of five projects were conducted, three of which focused on the C2 system design.

The findings of three of projects concluded that the C2 design concept was correct in many of its assumptions and provided good treatment to household wastewater prior to entering the groundwater table. It was also concluded that the imported fill, which is the hallmark of this design, could be cut back in height from 450 mm (18") to 300 mm (12") without compromising the system performance, thereby leading to substantial installation cost reduction for the consumer. The research also confirmed the maximum application rates of wastewater which could be applied to a N.S. soil and still allow the design to remain functional over the long term. This knowledge will allow future designs to be established with greater confidence in long term system reliability and fewer associated legal repercussions.

Site evaluation practices have benefited substantially from this research in that new and existing technologies were rigorously evaluated for their application to On-Site systems. The constant head permeameter has been seen to be the most effective, inexpensive method as a lot assessment tool prior to site development. Consultants and government authorities will both benefit from this standardized method of testing through both time savings and more reliable site data, both necessary for an efficient and cost effective means of on-site design.

Information acquired from New England has provided a useful background related to topics that include water use, reduction and conservation, Wastewater Management Districts, as well as system design and performance.

Recommendations of this document are found in each of the sub-reports and are now being considered for application in the revised N.S. Department of Health and Fitness design guidelines for On-Site wastewater treatment facilities. The implications of the results are summarized as follows:

Implications

In-Situ Evaluation of C2 Fields

- Existing C2 systems were monitored to determine their performance and yielded the following results:
- C2 septic fields have proven effective in removing BOD, COD, Organic Carbon and fecal coliforms from septic effluent.
- In fine soils exposed to high annual rainfall (~ 1200 mm/yr), separation distances between the distribution trench and the groundwater table can be in the order of 200 mm to provide adequate treatment of septic tank effluent.
- Nitrification is not occurring in the C2 systems or in the soil immediately adjacent to the system.
- Investigation of a C2 system serving a household which discharged the back-wash from its water softener into the septic field indicated hydraulic overloading leading to partial malfunction as well as extremely high Chloride levels (in excess of 4000 mg/l) in the surrounding groundwater table.
- To prevent the distribution trench from back-flowing to the interceptor drain, a minimum separation distance of at least three meters is required.

Methods of In-Situ Permeability Measurement

- The constant head permeameter is the most reliable method for testing the global in-situ permeability of a small lot and is recommended for use in on-site assessment. However, values obtained may be slightly higher than the true global permeability of the field.
- The shallow well pump-in test is too labour intensive and does not provide a means to adjust for differing soil conditions unlike the constant head per-meameter (see figure 4). This test is recommended only if the constant head apparatus is unavailable. In providing a final analysis, the equation governing the constant head permeameter should be used.
- To establish a reliable estimate of a given lot, a minimum of five tests is recommended (ideally in the region most likely to support the future septic bed).
- In using the constant head permeameter, it is necessary to penetrate the ground, well below the humus and root zone.
- The double ring infiltrometer, as documented in this study, appears to be too unreliable and cumbersome for use in the assessment of on-site permeability and is not recommended.

Clogging Mat Hydraulics and Treatment

- The most significant implication of this work is a need to provide adequate diffusion of gases to and from the clogging mat. If a mechanism for gas movement is not provided, a dominantly anaerobic bacterial biomat will develop which fails to effectively treat COD and BOD levels of septic wastewater, and develop extensive clogging of the host soil. Although deply constructed septic wastewater disposal trenches may not provide oxygen gas exchange into the clogging mat , shallower, near surface designs would. Furthermore, both deep and shallow designs will permit gasess to diffuse laterally away from the clogging mat and not inhibit wastewater flow. Area bed designs could possibly function in a manner similar to the columns, as the trench area (paricularily in the center) would not provide any pathways for oxygen exchange into the subsurface. This could explain why many older area beds installed in tight soils failed so readily. Design of any experimental system should also consider this problem of gas exchange to minimize the effects of developing biomat environments which differ substantially from a true in-situ contour trench condition (ie aerobic vs anaerobic).

- A second implication of this work is that of the mobility of bacteria into the groundwater table should it rise to a height nearer than 300 to 450 mm from the bottom of the trench.

Hydraulics of Imported Fill Systems

- Adequate treatment of septic wastewater can be achieved in an imported fill section whose depth is 300 mm. This represents a 50% reduction in the amount of imported fill currently specified under the N.S. Dept. of Health and Fitness Guidelines.

- If 50% of the incoming effluent does in fact enter into the imported fill region, there exists a brief three month period following initial construction, in which this type of design is capable of discharging harmfull bacterial groups from its toe region, particularily during periods of high moisture (eg extended periods of rain, flooding etc). (Further research regarding this topic is under examination during the publishing of this report)

- Given adequate controls on incoming moisture such as rain, groundwater, septic loading, a 1.6 m long by .3 m deep imported fill section is quite capable of treating at least 50% of a 200 gal/day loading (the operating volume of the model). Extending the results of these findings to higher loading conditions may provide the basis for a new in-situ wastewater disposal design which utilizes only the imported fill section in providing treatment. Such a sand filter design could be used in the difficult soils found extensively throughout Nova Scotia.

On-Site Related Information From New England

As a result of talks and discussions with various officials and consultants, useful information has been gathered relating to the following:

- Wastewater Management Districts
- Water Conservation
- System Regulations
- Design
- Performance

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N.S. Department of Municipal Affairs

N.S. Department of Agriculture and Marketing

N.S. Department of Industry Trade and Technology

Laboratory Modelling of C2 Imported Fill Sections

Wastewater Hydraulics and Chemistry

G.M. Charles, D.H. Waller

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Introduction

Imported fill contour disposal fields are currently the accepted standard for use in Nova Scotia. Figure 1 illustrates a typical contour field in plan and section. The contour field is a single pipe laid in a gravel filled trench which follows a single contour elevation. The imported fill design also includes a mound soil placed over the pipe to increase the effective depth of soil.

Imported fill contour fields are used in the following situations:

- where there is insufficient depth of permeable soil over impermeable subsoil
- where the topography and dimensions of a site preclude the installation of a full length contour trench
- Where the permeability of the natural soil is too low or there is an insufficient depth of low permeability soil over fractured bedrock or coarse gravel which would allow effluent to reach groundwater without adequate filtration.

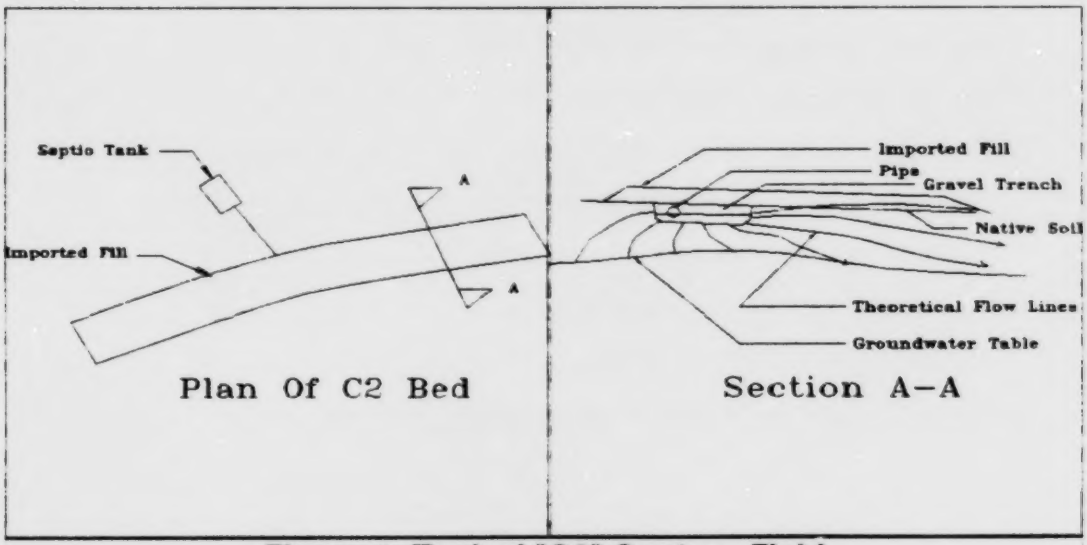

Figure 1. Typical "C2" Contour Field

Objectives of Study

The primary objectives of this study were as follows:

- to determine the hydraulic characteristics of effluent through an imported fill bed with respect to the distribution and magnitude of flow through zones of full and/or partial saturation;
- to evaluate the performance of imported fill (sand) with respect to the removal and/or treatment of potential pathogens (chemical and bacterial) and their extent of movement.

- to determine the effects of rainfall on the quality of effluent removal within an imported fill bed.

Methodology

To investigate the treatment and soil hydraulics of an imported fill bed, it was necessary to isolate a specific section through the septic bed. A model was set up to duplicate the region of imported fill located immediately downslope from the wastewater distribution trench, extending for a distance of 1.6 meters. The model was fed with a septic effluent collected from a single family dwelling. A series of tensiometers were installed along the wall of the model to visually evaluate the degree of soil saturation. To meet the objectives of the project, the study was conducted in four phases:

1. Pre Wastewater Soil Hydraulics : this phase was used as a control, evaluating soil properties prior to any modifications from septic wastewater. Measurements included soil moisture tension, volume of effluent retention, and velocity of lateral flow using a tracer dye.
2. Septic Tank Effluent: this phase utilized secondary effluent collected from a single family dwelling. When necessary, the effluent was adjusted to maintain minimum COD levels so as to ensure reasonable rate of clogging mat formation. Tensiometer measurements identified any influence of the clogging mat on the soil hydraulics and variation of water flow within the model. The outflow was collected once weekly to evaluate the degree of treatment.
3. Effects of Vertical Water Infiltration: this phase occurred six months after the septic effluent dousing had begun. Two 20 mm rain events were simulated using a perforated hose which ran across the top of the model. The duration of each rainfall was varied. During both tests, outflow samples were collected as well as soil tension levels.
4. Long Term Soil Hydraulics: The final part of the experiment was an assessment of long term soil hydraulics using (i) steady state and (ii) slug test conditions. This final phase was necessary owing to difficulties in assessing the soil hydraulics using transient inflow conditions.

Description of Model

An isolated section was contained in a clear acrylic channel 150mm x 450 mm x 2.0m in length. The channel was in turn supported by a wooden timber frame which inclined the model on a 10% grade. Forty-nine tensiometers, consisting of 6.25mm x 3.13mm clear PVC tubing fitted with non-woven geotextile wicks, were installed at various locations along the walls and bottom of the model. The tensiometers were positioned in seven vertical rows, with six up the channel wall and one located along the center line of the bottom. The free ends of the tensiometer tubing were grouped on measuring scales mounted on each of the three legs of the frame. Provision for distribution of the effluent was made possible by placing 150 mm of gravel at the upslope end. A section of sand, equivalent to the imported fill extended for 1.6 meters where it again interfaced with a 100mm gravel layer. Effluent was fed to the system through a 19 mm x 13 mm clear acrylic inlet pipe buried in the gravel to a depth of 100 mm.. Effluent exited the model through a 8mm diameter PVC pipe located at the downslope end. Figure 2 illustrates the relation between the model to an actual imported fill septic field.

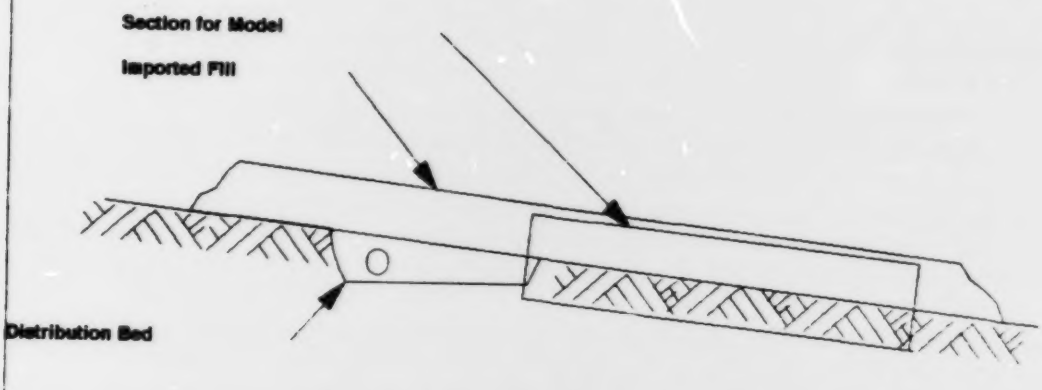

Figure 2. Relationship Between Model and Field

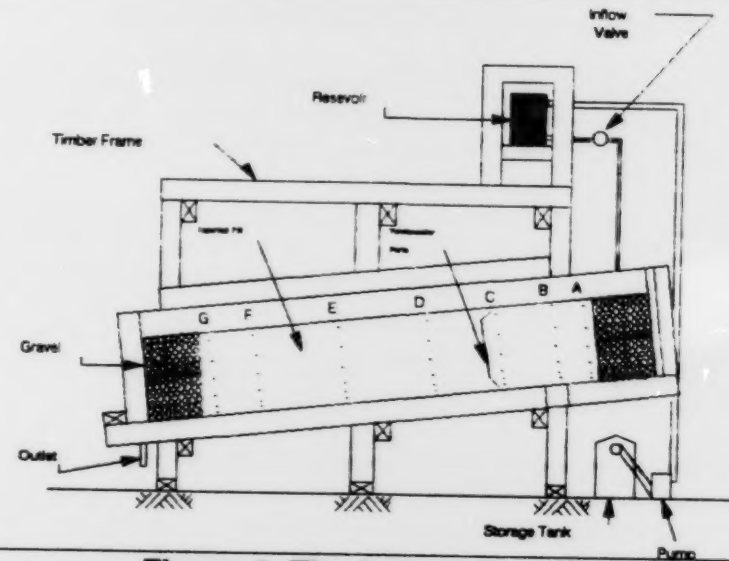

Figure 3. The Lateral Flow Model

During the first three phases, liquid was introduced to the model three times each day at a rate of 740 ml, or 2.2 liters per day. To mimic the daily loading sequence of a typical single family dwelling, the incoming wastewater was applied at 4:30 a.m., 10:30 a.m. and 4:30 p.m. every day using an automatic control valve located on the inflow reservoir.

Unsaturated Flow Phenomenon

As the operation of the imported sand fill is controlled by unsaturated flow hydraulics, it is necessary to understand the inter-relationship between the soil hydraulic conductivity, soil moisture and soil tension. Soil moisture tension is defined in figure 4. The total head can be described using the elevation of the manometer tip above a random datum plus the pressure head (which in the case of unsaturated flows is negative in value). Figure 5a shows the relationship between conductivity, moisture and tension for a typical silty sand. It can be seen that as the moisture content drops from the saturated value of 0.3 m^3/m^3 the hydraulic conductivity and suction head also drops. It is important to note, however, that there does exist a region on the curve which allows negative tension to be present and still maintain saturated soil conductivity. This so called "tension saturated state" will vary from soil to soil and will, in general, be less notable in sandy soils than in clay or very fine soils. The use of soil tensiometers then provides a tool which gives a crude assessment of the path and rate of soil moisture

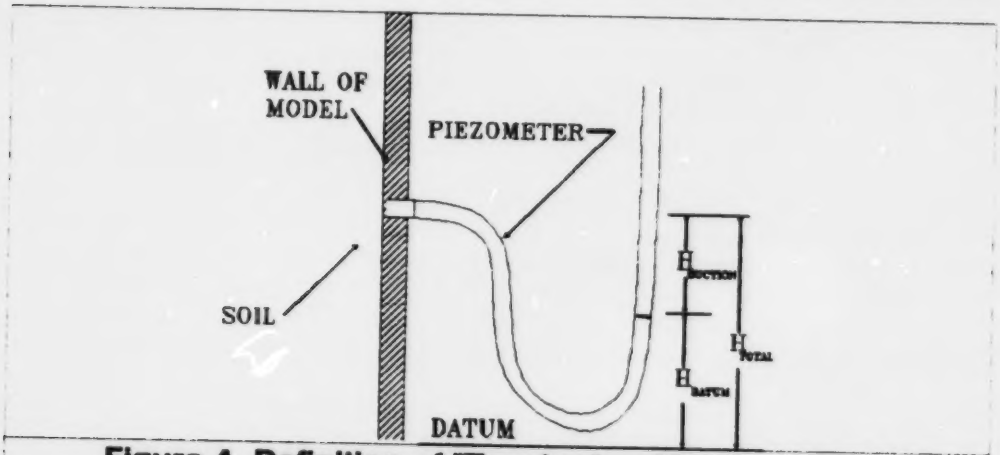

Figure 4. Definition of "Tension" or "Suction" Head

movement. In unsaturated flow, Darcy's law is still assumed to apply, (ie, the net direction of movement is dictated by the total head of a given point in relation to surrounding soil heads) but the flow rate is controlled by the degree of saturation. Figure 6 shows how the use of these tensiometers is capable of identifying vertically upward or downward moisture movement as a function of total head.

Procedure

Phase 1 - Hydraulics of Sand Bed

Before the bed was initially dosed with uncontaminated water, each of the manometers was primed. A phylam green dye solution was injected into the tensiometer tubing until a 5 to 8 cm diameter area of soil surrounding the tensiometer tip was wetted. The water column in each of the tensiometers was allowed to fall to a constant level, held by the negative pressure of the wetted area. The difference between the level in the water column and the elevation of the tip would be equal to the negative head tension (h_t in mm of water). Initial levels were recorded.

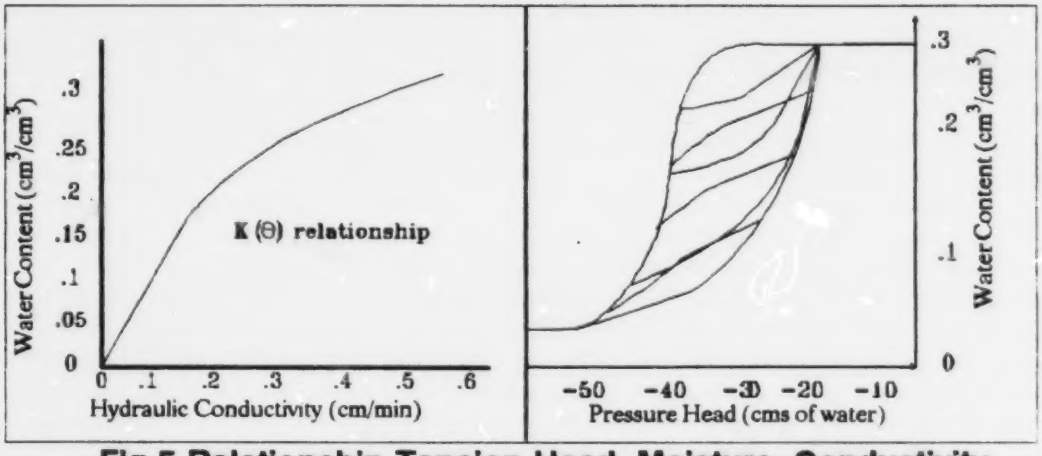

Fig.5 Relationship-Tension Head, Moisture, Conductivity

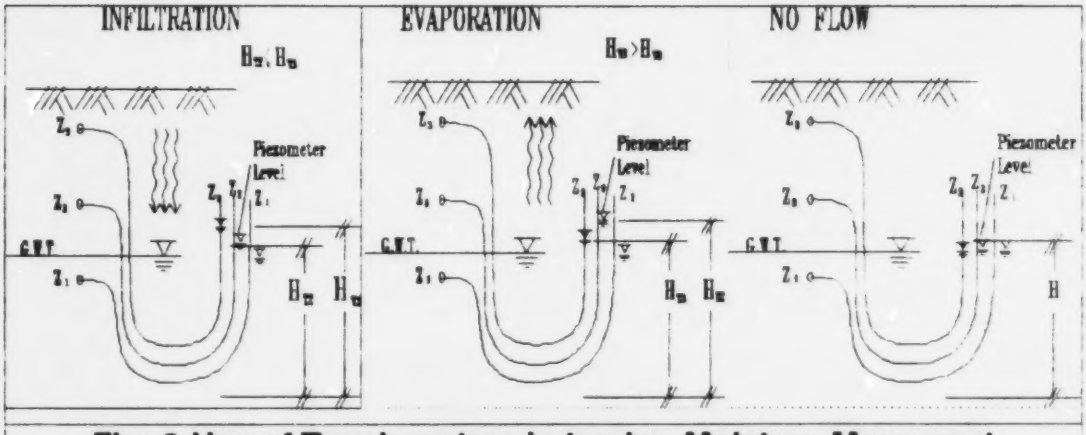

Fig. 6 Use of Tensiometers in tracing Moisture Movement

The bed was dosed with water at a rate of 2.2 liters per day. To simulate conditions of a normal household system, successive applications of liquid (730 mls) were applied three times each day at 4:30 am, 10:30 am and 4:30 pm, allowing a twelve hour rest period to occur during the evenings. This phase continued until steady-state conditions prevailed (i.e. inflow equaled outflow and the tensiometer levels remained relatively constant). A dye solution was then added to the influent to visually document the progress of liquid through the model. The dye selected for this purpose was Phylam fluorescent yellow mixed to a 2 g/l concentration. A series of photographs were taken at various times throughout the first week. During the second week, the dye was flushed out with clear water.

Phase 2 - Septic Effluent

After two more weeks of clear water application, allowing the dye to be flushed from the system, the septic wastewater was introduced to the model in the same manner as in Phase 1. For the first two months of this phase, one sample each of influent and effluent were taken weekly from the model. Chemical analysis included Biological Oxygen Demand (BOD), Chemical Oxygen Demand (COD), Total Organic Carbon (TOC), Ammonia, Nitrate-Nitrite and Total

Kjeldahl Nitrogen (TKN). Bacterial sampling was also carried out to evaluate levels of Fecal Coliforms. The soil piezometers were also monitored once each week.

Phase 3 - Vertical Water Infiltration

After five months of passing secondary septic effluent through the model, a 2.0 meter length of 25 mm diameter perforated pipe was placed over the top of the model. Two simulated rainfall events were run during which 20 mm of rain was applied over a ten minute period. During each rainfall, bacterial and chemical samples were collected along with outflow rates. Collection and monitoring was modified during the second rainfall event in order to capture outflow samples during peak flows. An outflow hydrograph was then created, comparing the two events. In both rain events, the tensiometers were read to document changes in moisture content in higher soil strata as well as the lowest strata where most of the effluent flow was expected to exist. Timing of the data recording is documented in Appendix B(i).

Phase 4 -

i) Steady State Hydraulics

An assessment of the model hydraulics was made eighteen months after the initial start-up. The steady state dynamics of the soil hydraulics were evaluated by setting up a continuous drip feed inflow device. During this one week period, the model was loaded with septic wastewater at a rate of 90 ml/hour (2.2 litres/day). Following one week of operation on the drip feed device, the soil tension was recorded as well as the head of wastewater retained in the upper gravel reservoir.

ii) Pulse Input Loading

Immediately following the steady state drip feeding, a single 2.2 litre instantaneous load was applied to the upper reservoir. During this period, a modified tipping bucket rain gage was placed at the lower end of the model to permit a continuous record of outflow. Single pulse counts emitted by the tipping bucket device were recorded by an electronic data logger. Due to technical difficulties, data was only recorded for a four hour period. Soil tension data was also collected at several periods throughout this phase, the timing for which is found in Appendix B(ii).

Observations

Dye Testing

The progression of the dye plume was documented through a series of photographs and is summarized in figure8. Using direct measurements, the lateral dye front progression rate was plotted as a function of time (figure7). By plotting the velocity of the plume front with time it was found that the effluent travelled at a diminishing rate during the first ten hours over a distance of approximately 600 mm and became constant at 30 mm/hr. Unfortunately, the exact moment of the dye discharge from the model was not visually documented but can be reasonably approximated at 55 hours based upon figure 2. This same retention time was consistent with records obtained from the same model in 1987. The vertical capillary draw of the dye varied greatly within the model due to stratification created by soil compaction. It was therefore not possible to accurately assess the vertical component of the dye front. The maximum vertical extent of the dye plume was approximately 130 mm above the floor of the channel.

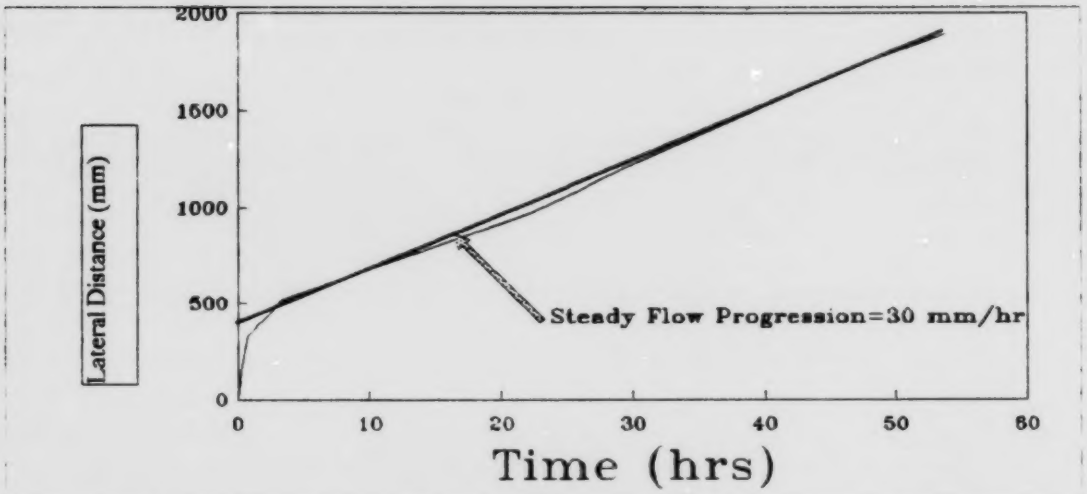

Figure 7. Progression of Dye Plume Front Over Time

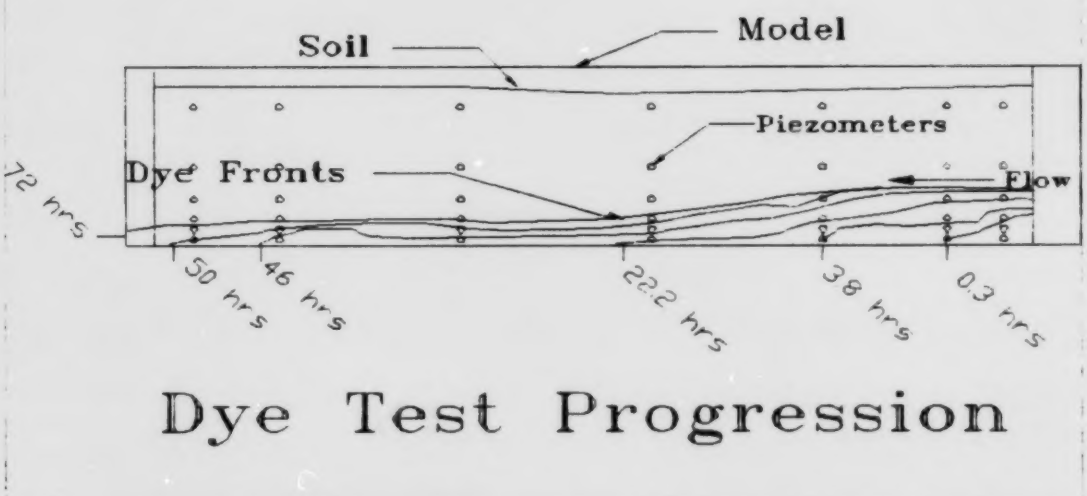

Figure 8. Progression of Dye Front

Soil Tension Profiles

The tension profile was constructed on an interpolation contouring package which uses kriging to prepare an equapotential mapping of the model. Profiling was compiled on several events during the experimental period.

Initial Soil Tension Prior to Wastewater Application

During the initial period of model operation the profiles established a fairly stable condition and is shown in figure9. It can be seen that the pressure head distribution was fairly linear and ran relatively parallel to the floor of the lab. Anomalies did exist near the upslope end where the soil was being repeatedly wetted throughout the influent discharge cycle.

Soil Tension One Year After Wastewater Application

Figure 10 shows the soil tensions twelve months after the wastewater was applied. Comparing the profile with the previous one, it can be seen that the downslope conditions did not change significantly with time. However, the tensions in the upslope region did change slightly becoming less negative, indicating higher soil moistures within the region of the clogging mat.

Soil Tension During Maximum Outflow of Rain Events

Figure 11 shows the soil tension profile during the peak outflow of rain event 1. The profile is not significantly different from that of Figure 10 and does not show any areas of positive soil pressure, indicating that even during this brief high intensity 20 mm rain event there was no areas of saturated flow. It should be noted however, that a saturated conductivity may exist in the regions where tension is greater than (more positive) -100 mm (refer to figure 5 re:Saturated Condutivity vs Tension Head).

Soil Tension During Slug Input

During peak slug input outflows, the tension profile (figure12) in the lateral flow model did not differ from that of the steady state conditions. Only in the uppermost region, where the water was applied, did significant variations exist. Though only marginally different from figure 10, the lower soil tensions do reflect the fact that a significant amount of water was present in the upper gravel reservoir (approx. 40 cm total head) and moved readily over the top of the clogging mat region (approximately 20mm in height). In a manner similar to the rainfall events, this test showed little change in the tension profiles and at no time was there any positive pressure head.

Water level variations in the upslope reservoir

Though exact records were not kept of water retention in the inflow gravel region of the model, it was noted that retention times did vary significantly over time. During the initial four months of operation in which septic wastewater was not applied, the water would remain ponded no more than one hour. However, eighteen months after applying wastewater, there was a continuous volume of liquid retained in the gravel trench which never completely drained (often maintaining a height of 15 cm six hours after the 4:30 am inflow). This phenomenon was believed to be a direct result of clogging mat formation.

Vertical Water Infiltration

Hydraulics

The outflow from the two rain events was plotted as a function of time and is shown in figure 13. The peak outflow appeared to occur within two hours after water was applied and the entire outflow hydrograph extends up to 25 hours after the initial event. Appendix C provides the spreadsheet summary of the apparent storage within the model during each of the rainfall events. Rain event one shows a storage of 2 mm (over the 25 hour period) while event two shows a net system storage of 5 mm. It is possible that drier soil conditions, more readily influenced by evaporation, prompted an increase in trapped capillary water.

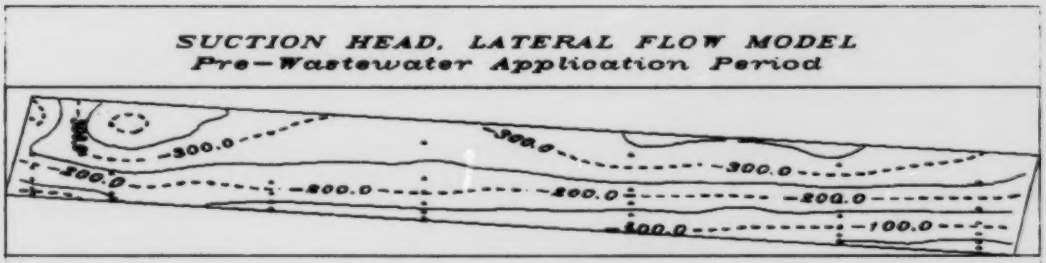

Figure 9. Soil Tension Prior to Wastewater Loading

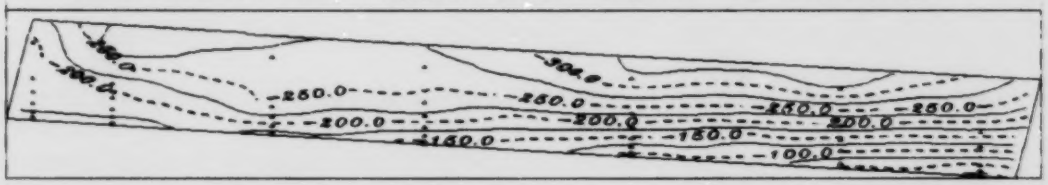

Figure 10. Soil Tension 12 Months After Wastewater Loading

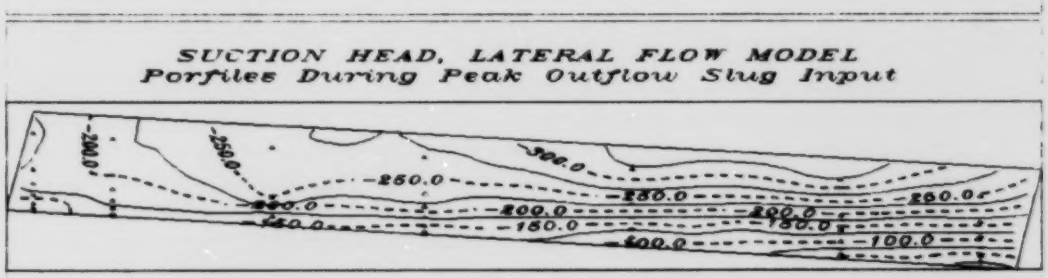

Figure 11. Soil Tension After Slug/pulse Input Loading

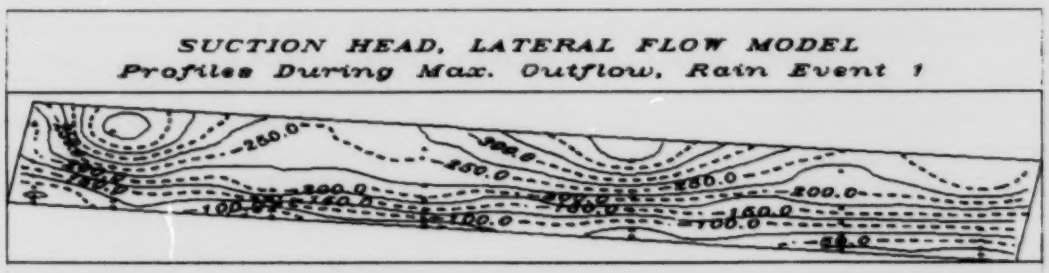

Figure 12. Soil Tension During Peak Rainfall Loading

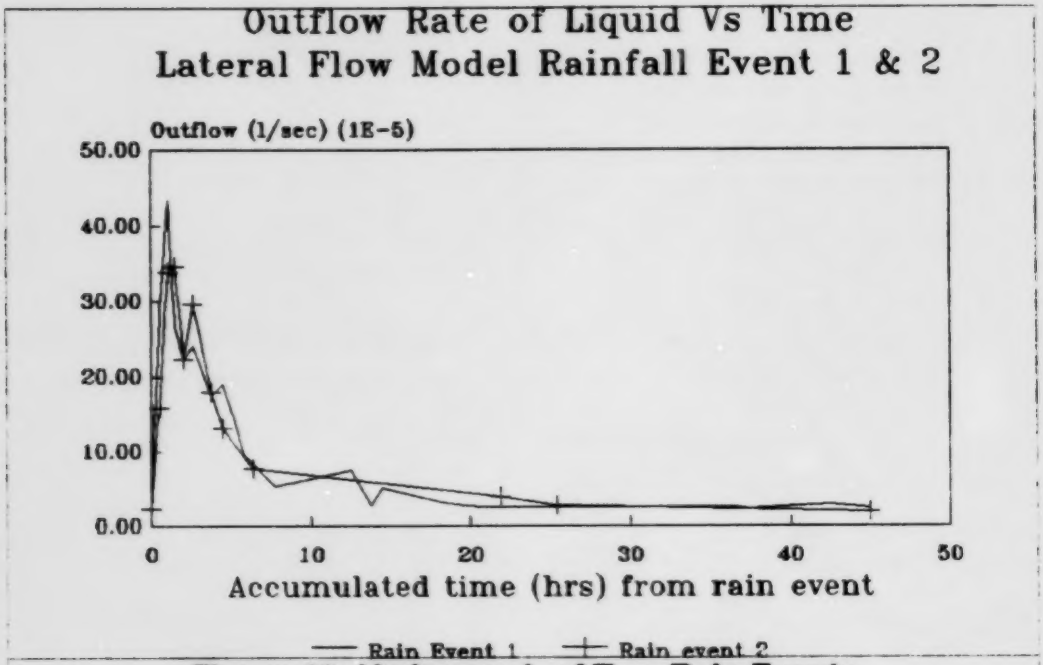

Figure 13. Hydrograph of Two Rain Events

Constant Inflow and Slug Testing

During the steady state inflow, the wastewater remained ponded in the upslope gravel bed at a level of 150 mm. The slug test was applied one week after the steady state conditions were developed, raising the level of ponded water up to 400 mm above the channel floor. The outflow from the first sudden surge peaked four hours after the initial application and stabilized 24 hours after the initial application.

Chemical and Bacterial Analysis

Nitrate-Nitrite Analysis

Nitrate-nitrite levels had consistently fell below the 0.05 mg/l threshold limits of the analytical equipment during much of the eight week period. Figure 15 shows the long term treatment of nitrate over time. It can readily be seen that during the early phases of the models operation, that the production of Nitrate was virtually zero. Four months after the septic wastewater application was started, the nitrate levels began to increase. The peak concentrations of nitrate occurred in late April of 1989, almost one year after the wastewater phase had begun. Afterwards, the nitrate levels dropped to a more stable level of approximately 50 mg/l. It was also noted that nitrate increases corresponded to a decrease in ammonia outflows.

Rain Events

During the rain events, nitrate levels remained relatively stable. Appendix F(i) shows the nitrate variations for both events. A greater number of samples were taken during the second event to identify any increased nitrate outflow levels during the peak discharge, though none was detected.

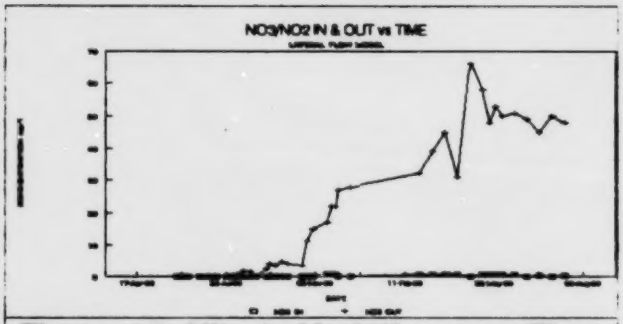

Figure 15. Nitrate Variation over Time

Total Organic Nitrogen and Ammonia

Inflow levels of total organic nitrogen was typically low, in the order of 10 mg/l. Most of the inflow nitrogen was in the form of ammonia. The TON levels were relatively high for a brief period two months after wastewater application began, and the removal was often quite high. As the model continued to operate, ammonia production began to decline, with an increase of nitrate outflow. Ten months after the model began accepting wastewater, ammonia levels dropped to zero and remained at that level until shutdown.

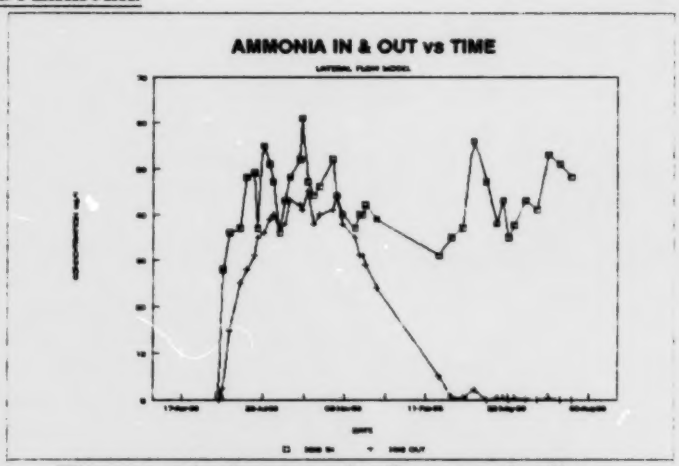

Figure 14. Ammonia Concentrations (Inflow/outflow) Vs Time

Rain Events.

The outflow levels of ammonia were similar to that of nitrate in that the levels were quite stable even during the peak outflows.

Total Organic Carbon

TOC inflow levels were widely varied over time ranging from 40 mg/l to 110 mg/l. The outflow levels were more stable at approximately 10 mg/l. There was vary little difference in the degree of TOC removal over the time of the project.

Rain Event

The TOC levels in the outflow did increase during peak outflow to values equivalent to the inflow but fell quickly back to the 10 mg/l level (within one hour).

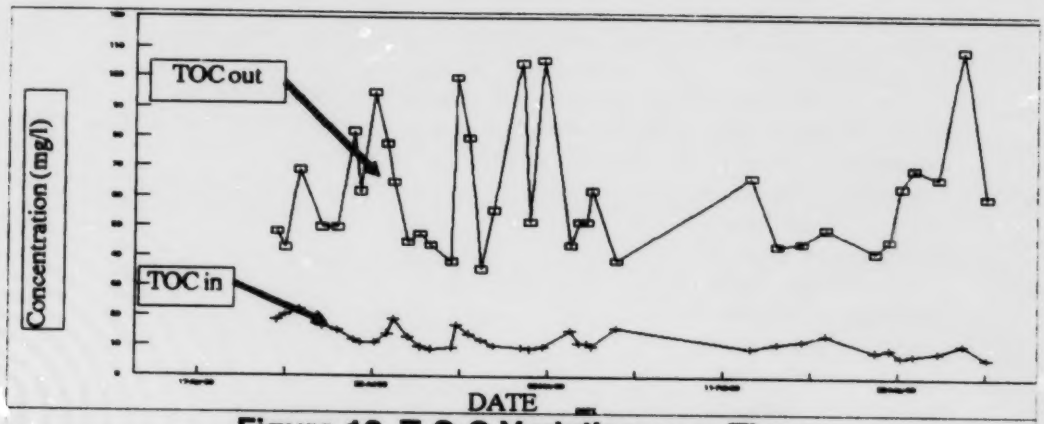

Figure 18. T.O.C Variation over Time

Chemical and Biological Oxygen Demand

COD and BOD removal was quite high over the time of this project. COD outflow levels showed a slight increase during the initial month of operation but quickly stabilized to a mean value of 50 mg/l. The inflow concentration of COD was on average 300 mg/l while the BOD averaged 110 mg/l.

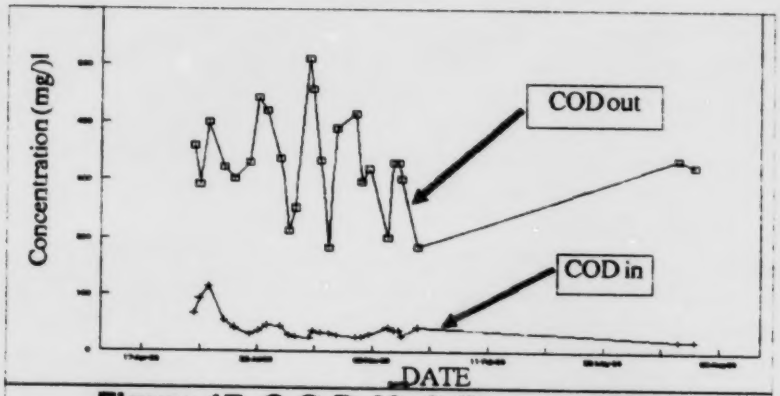

Figure 17. C.O.D. Variation over Time

Rain Events

Attenuation of COD and BOD were quite stable during rain events. Neither showed any increase during peak flows.

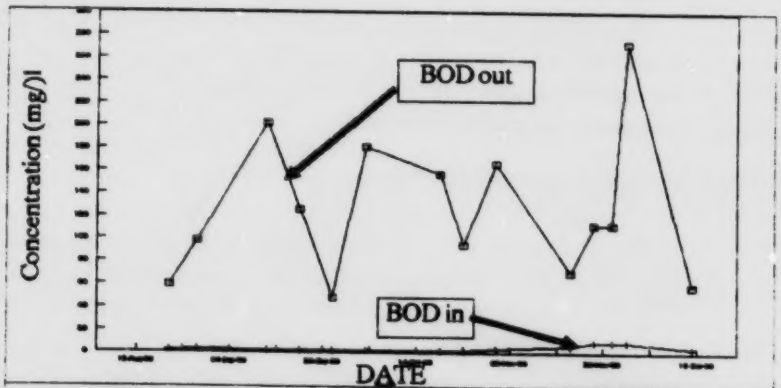

Figure 16. B.O.D. Variation over Time

Fecal Coliforms

Inflow levels of Fecal Coliforms were highly variable over the life of this project. The inflow levels were particularly high during the first six months of operation with counts reaching 3 million mpn/100 mls. The outflow showed a similar variation though the levels diminished with time. There were intermittent periods throughout the study which saw unexpected fecal coliforms passing through the sand bed through to the outflow. The maximum fecal coliforms discharge occurred two months after wastewater application began, reaching values of 5000 mpn/100mls. Soon after this peak discharge of fecal coliforms, the model showed continued declines in fecal coliforms outflow. During the last several months of operation, no fecal coliforms were detected in the model outflow.

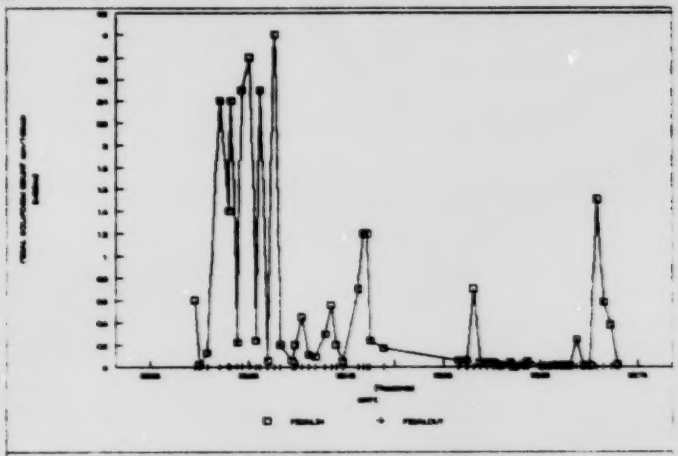

Figure 19. Fecal Coliforms (Inflow/outflow) Vs Time

Rain Events

Fecal coliforms counts in the outflow increased slightly during rain event 1 from zero to ten mpn/100 mls during peak outflow. However, during rain event two, the peak outflow count reached 900 mpn/100 mls. One hour after the peak discharge, the fecal coliforms count dropped back to zero.

Discussion

Soil Hydraulics

The long term hydraulics appeared to be modified only where clogging mat development occurred. In the region of the clogging mat, there appeared to higher levels of moisture as indicated by the lower tension head found in the piezometers. As the clogging mat appeared to create a resistance to flow, the incoming wastewater would pond in the gravel reservoir likely "spilling" over the top of the existing mat. This had the net effect of promoting vertical clogging mat formation at higher points in the sand bed. In regions more distant (downslope) from the clogging mat, the soil showed a more stable, horizontal soil tension profile, indicating the moisture contents were influenced more by other forces such as evaporation and lateral unsaturated flow. By close examination of the tension head profiles near the outflow end, it would appear that the zero tension elevation is concurrent with outflow tube elevation.

Figure 20 shows the long term variations which occured in the hydraulic gradient through the clogging mat over the eighteen month life of this project. It can be seen that the gradient increased 20 % from its original value of approximately eight. The relative increase was seen

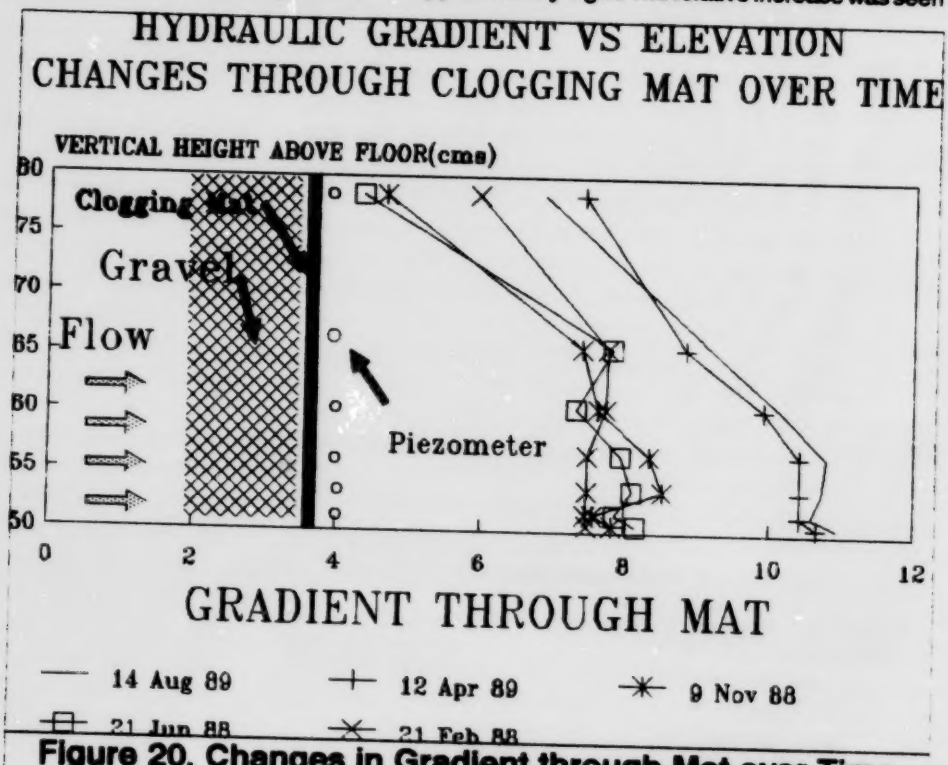

Figure 20. Changes in Gradient through Mat over Time

uniformly throughout the height of the model (450 mm). The increased gradient was accompanied by a decrease in the global permeability through the mat (given the increasing storage volumes observed in the gravel inflow reservoir over time) and is reflected in equation 1. During mat formation, decreasing soil conductivity was apparently coupled with an increase in the necessary contact region of the soil, promoting vertical clogging mat formation as more wastewater was contained in the upslope gravel reservoir.

$$i = 10 \text{ cm/cm}$$

$$A = \text{long term contact area of sand fill}$$

$$= 30\text{cm} \times 15 \text{ cm}$$

$$= 450 \text{ cm}^2$$

$$K_{avg} = Q/iA = \frac{2200\text{cm}^3/\text{day}}{10 \times 450\text{cm}^2}$$

$$= 0.5\text{cm/day}$$

$$= 5.6 \times 10^{-8}\text{m/s}$$

Equation 1:Effective Conductivity Through Clogging Mat

The soil in the model remained unsaturated at all times during the project. Even during the rain events, the sand showed little change in soil tension and did not become saturated along the lower boundary of the model. The unsaturated conditions made an accurate description of the soil hydraulics difficult as it profoundly effected the conductivity and

direction of flow in any direction. It is still uncertain if the soil along the lowermost soil horizon, bordering the channel floor, operated under saturated conductivity conditions even though the moisture tensions were negative. Testing of this soil was not carried out to develop a soil tension-hydraulic conductivity curve describing this phenomenon. However, it would appear that regardless of the fact that the model operated under unsaturated conditions, wastewater movement through the system was long enough to promote adequate treatment even through this short fill section.

Although the dye test indicated 52 hours was required to pass effluent through the 1.5 meter section, both the slug test and rain events showed increased outflow surges were capable of developing within two to four hours of an event (either a rain event or slug test). In the case of the slug test, it is expected that the wastewater applied was not influenced hydraulically by the clogging mat which had not formed higher than 150 mm above the channel bottom. Since the constant drip mechanism had ponded water up to the 150 mm mark, the slug surge had simply passed through the higher soil horizons and migrated readily through the soil matrix. Unfortunately, the soil tensiometers were not able to pick up significant changes in the upslope soil region, making it difficult to establish the true flow path of the slug test moisture movement.

The results of the dye test indicated that the maximum depth of the migrating effluent plume was approximately 150 mm. It is therefore resonable to assume that the current design height of 450 mm for the imported fill section could be reduced by 150 mm without compromizing the performance of the fill. This would result in a net decrease of 50% to the purchace cost of the imported fill during construction.

Chemical/Bacterial Removal

The overall performance of the imported fill in removing wastewater contaminants was quite good. The only significant problems were related to the production of nitrate/nitrite and the occasional occurrence of Fecal Coliforms in the outflow (particularly during the third month of operation).

The high production of nitrates were believed to be partially due to the experimental design. It was initiallyconjectured that some aeration of the septic wastewater did occur during the recirculation of wastewater to the upper storage reservoir. This pump method was discontinued after one year, resulting in only a slight drop in nitrate production. It is uncertain as to whether or not nitrate production is as severe in a true system understanding that at least half of the wastewater is capable of moving through soils of lower permeability and lower aeration. Emerging wastewater (assumed at the toe of the imported fill) would likely contact vegetation which would readily take up the nitrates before the plume passed into the groundwater. The ammonia levels dropped in a manner inversely proportional to the production of nitrate. Appendix G illustrates the nitrogen budget over the life of the project. As all nitrogen products were reported in terms of nitrogen, a mass balance was performed. The results indicate a 14% nitrogen deficit which is assumed to result either from the formation of undetected nitrogen gas or insufficient sampling (inflow/outflow measurements occurred weekly). One final possibility may also be in the ability for the sand to capture some of the ammonia nitrogen or organic nitrogen.

It is assumed that most of the treatment occurrs within the clogging mat and it is within this region that the flow hydraulics are most regulated. Earlier fecal coliforms removal tended to be much lower than later ones and could be attributed to the gradual formation of the clogging mat followed by possible "channeling" within the soil matrix or, more likely, the effects of "spilling" over the clogging mat. Spilling results from the clogging mat developing a sufficiently impermiable lower boundary, causing effluent to pond in the gravel bed. As a

result of ponding, the inflowing liquid would be forced to contact upper soil strata along the soil gravel interface and result in a more rapid, unimpeded travel path within the soil system. It is felt that if the system was left to equilibrate, the clogging mat would continue to extend vertically up the gravel soil interface until some equilibrium position was established.

Conclusions

The data collected from this model has revealed several important findings regarding the wastewater renovation capacity of the imported fill (based upon loadings of 1.5 ft^2/gal/day) and are summarized as follows:

- The imported fill section of a C2 contour field is capable of adequately removing harmfull pathogens from septic wastewater.
- Depending upon the degree of aeration in the sand fill, elevated production of nitrites/nitrates can be possible if sufficient wastewater enters this region.
- As the development of an effective clogging mat can take several months, there can be brief periods during which fecal coliforms can be mobilized and travel through 1.6 meters of the imported fill.
- The flow of septic wastewater appears to be confined to the lower 125 mm of soil in the imported fill section and the ability to treat effluent does not appear to be dramatically influenced by short periods of increased moisture.
- Soil moisture levels in the imported fill region do not become saturated due to incoming septic wastewater and therefore provides a good environment for reducing the contaminants and bacterial groups typically associated with sewage.

Implications

- Adequate treatment of septic wastewater can be achieved in an imported fill section whose depth is 300 mm. This represents a 50% reduction in the amount of imported fill currently specified under the N.S. Dept. of Health and Fitness Guidelines.
- If 50% of the incoming effluent does in fact enter into the imported fill region, there exists a brief three month period following initial construction, in which this type of design is capable of discharging harmfull bacterial groups from its toe region, particularily during periods of high moisture (eg extended periods of rain, flooding etc). (Further research regarding this topic is under examination during the publishing of this report)
- Given adequate controls on incoming moisture such as rain, groundwater, septic loading, a 1.6 m long by .3 m deep imported fill section is quite capable of treating at least 50% of a 200 gal/day loading (the operating volume of the model). Extending the results of these findings to higher loading conditions may provide the basis for a new in-situ wastewater disposal design which utilizes only the imported fill section in providing treatment. Such a sand filter design could be used in the difficult soils found extensively throughout Nova Scotia.

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Laboratory Modelling of Clogging Mats

Wastewater Hydraulics and Chemistry

G.M. Charles, D.H. Waller

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Introduction

Over the past five years, the Nova Scotia Department of Health and Fitness has adopted the "Contour disposal field" as the preferred design for on-site septic waste treatment. However, due to the diverse nature of the soils in Nova Scotia, three different types of contour design have been developed. Figure 1 illustrates one such contour field known as the "C2" system. The C2 contour field is for installation of septic fields in soils of poor permeability. To overcome the inadequate percolation of septic effluent into tight soils, an imported fill is placed over the natural soil to provide treatment of sewage which would otherwise flow upwards and travel overland. The underlying design philosophy of the C2 system maintains that migrating effluent introduced into a soil of low permeability and high groundwater will travel substantially in a lateral direction. Such fields must be located on terrains of 5 to 30% grade and follow one specific contour elevation. This design will then permit installation of septic fields in locations where the water table may be as close as 450mm below the distribution pipe.

Understanding the vertical soil hydraulics and wastewater treatment becomes critical when dealing with this new type of system and is the focus of this report

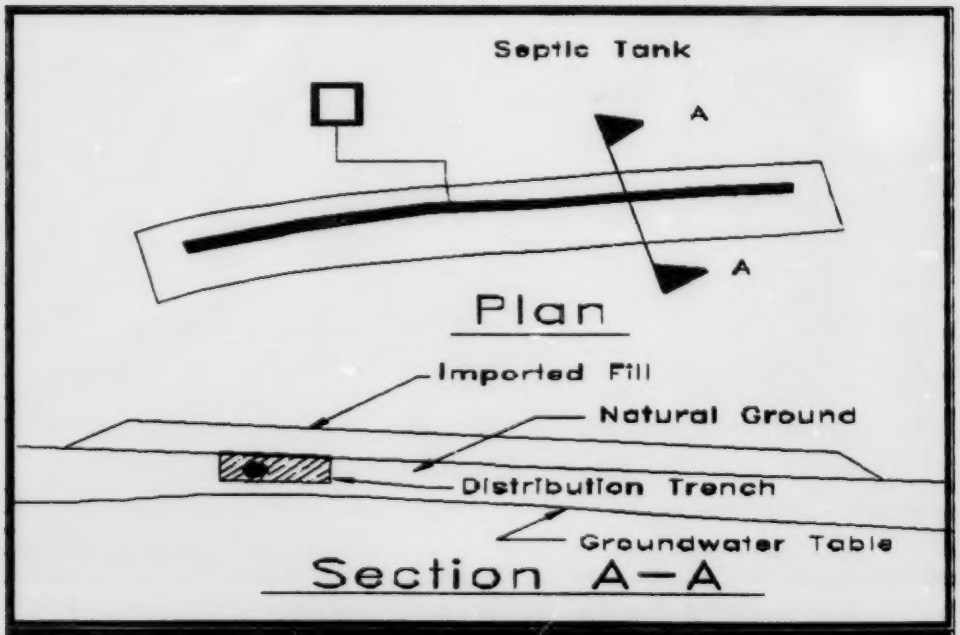

Figure 1. Typical C2 Contour Field

Objectives

The objectives of this research is to:

- To evaluate the influence of clogging mat formation on soil hydraulics over time.
- To evaluate the efficiency of pathogen removal in the vertical direction as groundwater fluctuates below the region of the clogging mat.

Review of Unsaturated Flow

In considering the operation of any septic field, it is necessary to understand some of the hydraulic principals which govern flow, particularly in an unsaturated soil regime

$$q = -K(z)\frac{d(h)}{d(z)}$$

Equation 1: Darcy Flow

where

q=flow (m^3/sec)
K(z)=Conductivity (m/sec)
h=head (m)
z=vertical distance above datum (m)

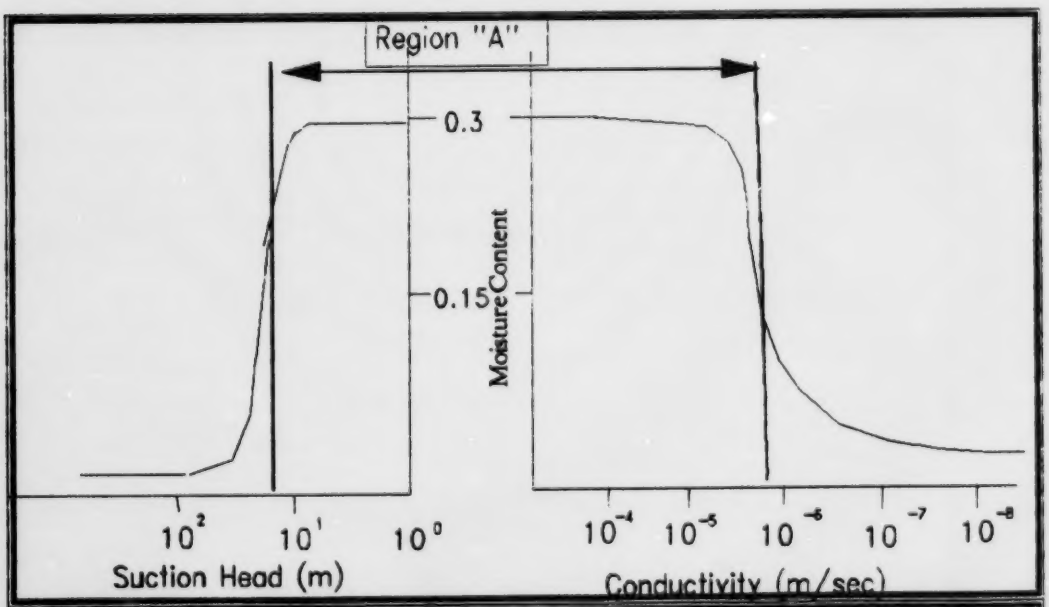

Figure 2. Moisture, Tension, Conductivity Relationship

Vertical flow can be represented by equation 1, the Darcys' flow equation. This equation can be applied to flow in either saturated or unsaturated soils. However, in dealing with unsaturated soils, the conductivity K, is dependant upon the degree of soil moisture content and pressure head. Unsaturated soils will always exhibit a negative pressure head owing to the capillarly forces found within the soil matrix. Figure 2 shows the relationship for a silty sand between soil moisture, hydraulic conductivity and pressure(or suction) head in unsaturated flow conditions.[1]

An important feature of these inter-relationships is that region where negative pressures or less than complete saturation exists. As moisture content declines, increasingly negative soil tensions modify conductivity to a point of no flow. One other feature of these curves is that region where negative pressure head is present to some degree, but the conductivity is still equal to that of a fully saturated soil. This region will often have negative pressure heads yet still support almost complete saturation and is known as the tension saturated zone. The columns operating in this study were most likely influenced by that region defined by "A" on Figure 2.

Procedure

Two 145 mm diameter acrylic columns, 1000 mm in height were packed with a silty-sand soil. In each column, a 150 mm layer of stone was placed both at the bottom and at the top. A geotextile was placed at the interface between the soil and the gravel at the bottom. of each column. Columns were compacted using a 1.2 kg tamping block in 150 mm layers. After placement, the models were subjected to positive water pressures which could be

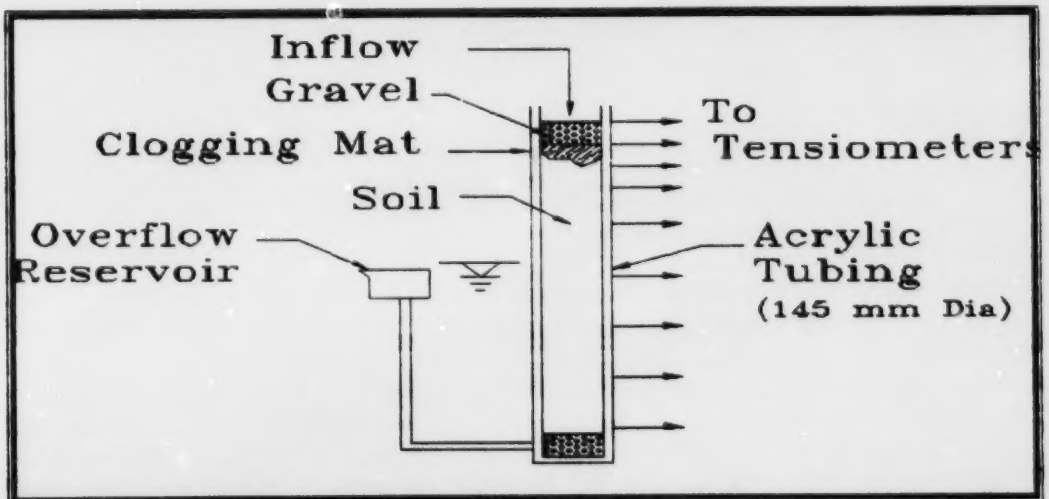

Figure 3. Typical Laboratory Column

1 Soil and Water, Physical Principals and Processes, Hillel, D, Academic Press, 1971

applied to the bottom of each column through a tube attached to a constant head overflow container, used throughout the experiment. Negative porewater pressures could be evaluated using tensiometers located down the side of each column. The constant head overflow device was set to four different positions during each phase of the experiment to determine the effects of the groundwater on suction pressures below the clogging mat and the ability of the soil to treat sewage effluent. Figure 3 shows the details of the column models.

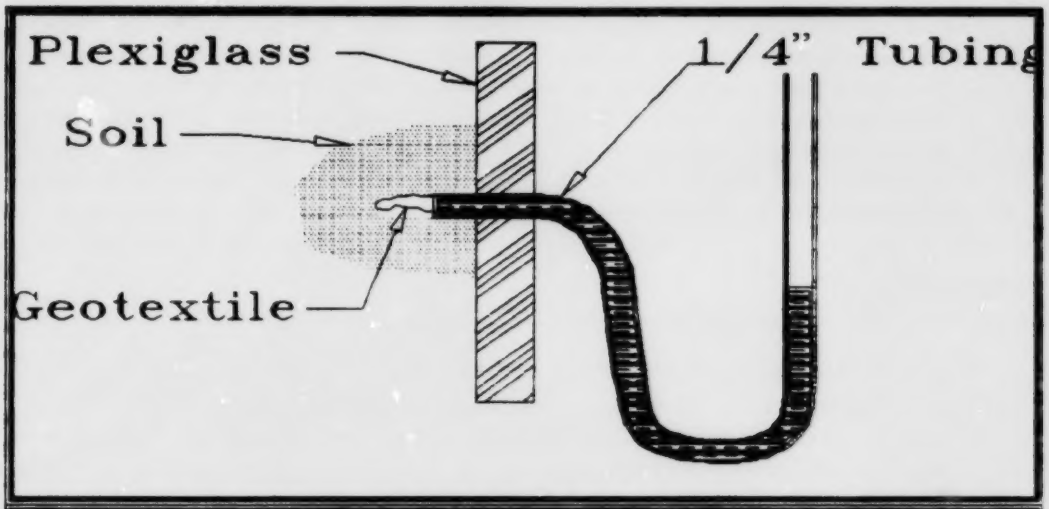

Figure 4. Detail of Piezometer

Methodology

This experiment was run in three different phases. A complete synopsis of the columns operation is found in Appendix A.

Phase 1

In phase 1, septic tank effluent was passed through each column to establish the typical hydraulic characteristics of the soil prior to clogging mat formation. Both columns received a specific volume of effluent, which was applied using an automated dosing apparatus. The effluent was added three times each day and was applied at the maximum field loading rate of 1 cm per day (1 ml/cm^2/day). The operating temperature during this period was 2° C. Throughout this 22 week period the water table was set to four different elevations, 1000, 600, and 300 mm and back to 1000 mm below the upper soil/gravel interface and the negative porewater pressures recorded. The first three data sets (1000, 800, 300 mm) were collected over a fourteen week period. The models were then allowed to run for eight weeks in order to further establish the clogging mat using the same dosing procedure. During this time, the groundwater table was set one meter below the clogging mat region.

Phase 2

Twenty two weeks after the initial start-up, the automated dosing procedures were modified to create a fixed head of effluent above the clogging mat and the columns allowed to accept wastewater as a function of the soil/clogging mat hydraulic properties. During this period, the water table was positioned within 600 mm below the clogging mat. After a further 10 days, the water table was re-set to within 300 mm of the clogging mat and the soil allowed to come to equilibrium. Three weeks later, the water table was allowed to come into direct contact with the clogging mat and remained in this position for two weeks. Over the following two week period, the water table was dropped to 300 mm, 600mm and then 1000 mm below the clogging mat.

Throughout all of Phase 2, weekly records of Total Organic Carbon (T.O.C.) Nitrate-nitrite (NOx), ammonia (NH_3)Chemical and Biological Oxygen Demand (C.O.D., B.O.D.) and fecal coliforms bacteria were maintained for both the influent and effluent of each column. Soil moisture tensions were recorded weekly.

Thirty-one weeks after start-up, the columns received a mixture of effluent and a modified lactose mixture to simulate effluent quality (COD levels equal to 400 mg/l). This modification was necessary because of a temporary four week shortage of domestic effluent (Staff on holidays, collection not possible).

Phase 3

Immediately after the three week application of lactose solution, only septic effluent was applied to the columns. After a review of the columns performance using a fixed head of effluent, the dosing method was changed (Oct. 14, 1988). For the remaining 14.4 weeks of operation, the columns used the original application method with a dosing schedule of three times per day, 1cm/day. The operating temperature of the lab was also raised to 22 degrees Celsius. However, due to extensive mat formation, the loading rate was cut back by 30%. The water table remained at a position 1000mm below the clogging mat.

Observations

Model hydraulics During Phase 1

Four months were required to visually establish the existance of the clogging mat. The clogging mat formation was seen directly below the upper gravel surface, appearing as a black region of soil having a lower bound area of greyish-brown colour. During this first four month period, the vertical columns exhibited erratic rates of septic effluent flow. Observations of the ponded effluent levels retained above the developing clogging mat showed continual change. Certain days would see up to 100 mm of ponded effluent, immediately followed by days of no ponding. Based upon these eratic ponding modes, it is possible that the clogging mat may have been influencing the hydraulics at an early stage of operation and only became "visible" after several months. Figures 4 and 5 illustrate the variation of suction head of columns A and B respectively throughout the experiment including the period of phase 1.

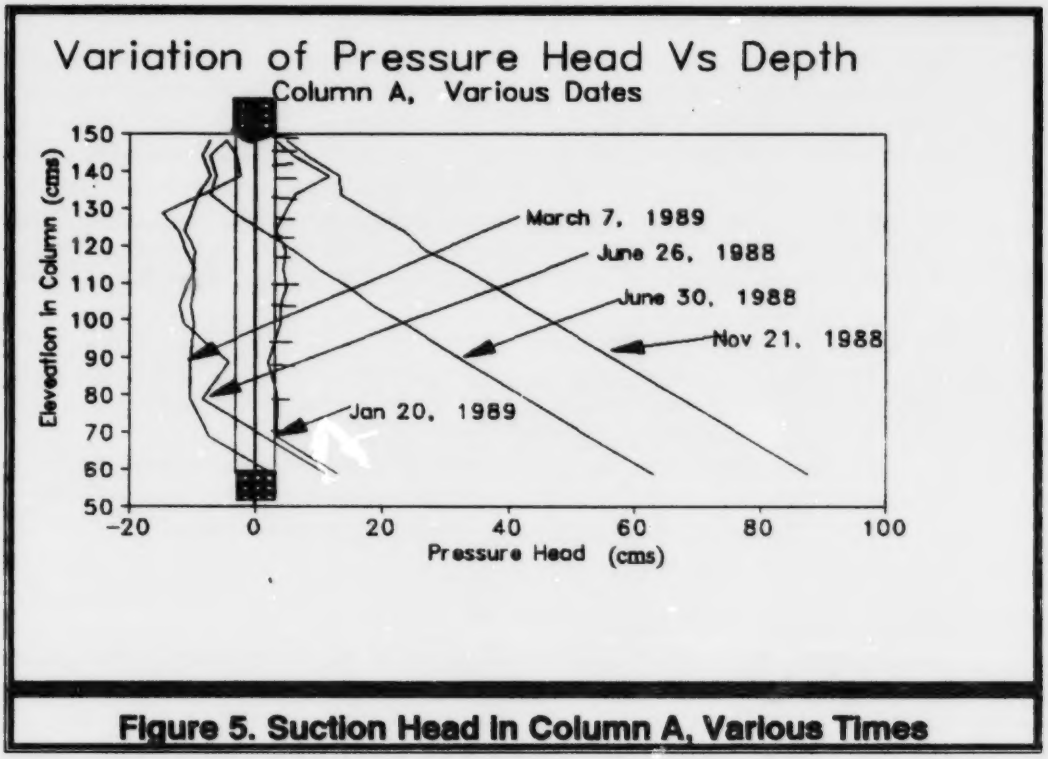

Figure 5. Suction Head in Column A, Various Times

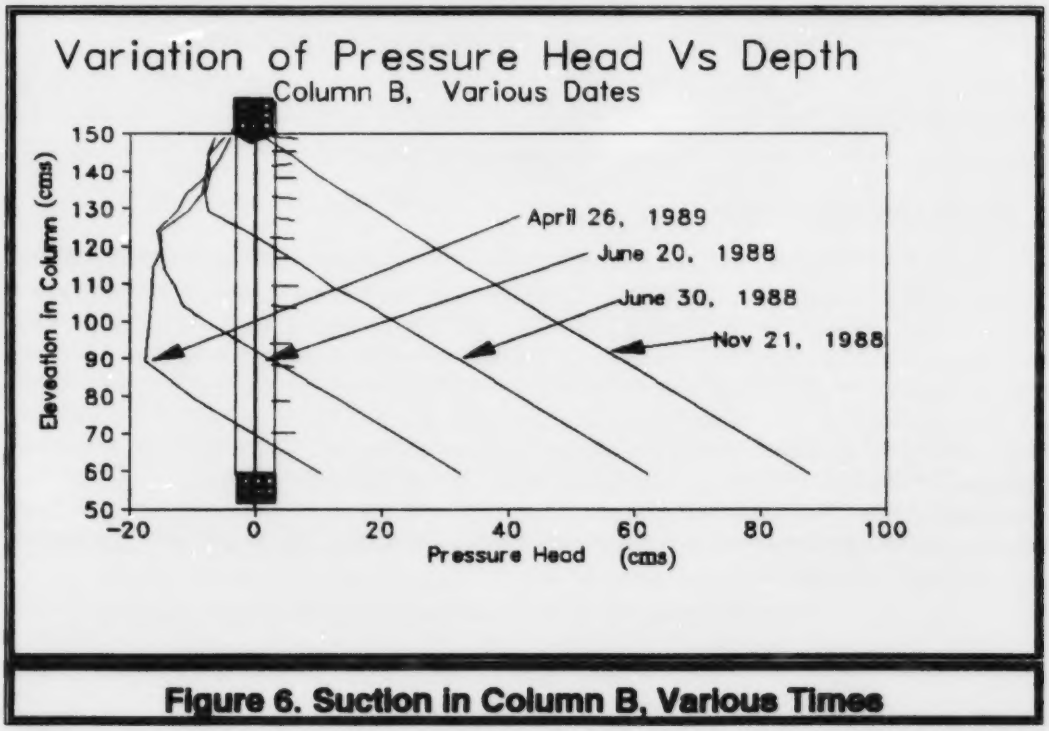

Figure 6. Suction in Column B, Various Times

Model Hydraulics During Phase 2

With the establishment of a 7 cm perched water table above the visible clogging mat, the variation of total head in the piezometers continued to fluctuate slightly, but the rapid day to day changes seen prior to this, stopped. However, within two months, the same phenomenon began to redevelop in which septic wastewater would move rapidly through the columns. During the times of high flows, the total head of the upper piezometers increased, associated with a notable decrease in hydraulic gradient. The rapid increase in flow would usually result in a complete draining of the wastewater inflow reservoir, allowing the clogging mat to come into contact with the atmosphere. During the times of high flows, the total head of the upper piezometers increased, associated with a notable decrease in hydraulic gradient. The rapid increase in flow would usually result in a complete draining of the wastewater inflow reservoir, allowing the clogging mat to come into contact with the atmosphere. During the period in which the lactose solution was added, extensive clogging mat formation had occurred which extended approximately 100 mm into the soil.

Model Hydraulics During Phase 3

Both columns developed considerable clogging mats by Phase 3. The dosing rate of 1cm per day was forced to be reduced by 30% due to overflowing at the column tops. At the reduced loading rate, both columns appeared to adequately move the applied effluent. However, niether column A or B permitted all the effluent to pass through the clogging mat before the next dosing occured, promotining anaerobic conditions in the clogging mat region.

Variation in Soil Saturation

Both columns showed a notable variation in the saturated/unsaturated profile over time as a function of watertable position. Figures 4 and 5 show how the pressure head (and therefore soil moisture content) varied with depth. After January 3, 1989, column A showed complete saturation with all piezometers indicating a positive pressure head at each location (even though the groundwater table was set at 90 cm below the clogging mat); this was not the same for column B. Eventually, column A became unsaturated as the in flow rate diminished (30%). Though column A exhibited a completely saturated state, the flow rate of effluent remained much lower and stable than periods prior to December 13, 1988. Column B also maintained a stable flow rate which was much lower than the period prior to December 13.

Chemical and Bacterial Attenuation

COD and BOD

Through the initial four months of operation, both columns exhibited good removal rates of COD and BOD (within the order of 80 to 90% removal) as illustrated in figures 6 and 7. After imposing a perched water table (phase 2), both columns exhibited a declining ability to remove COD and BOD. The two months following the imposition of a perched water table, both columns showed a maximum COD and BOD removal rate of 40%. Neither the location of the groundwater table nor the flow rate bore any direct influence on the removal efficiency of either the BOD or COD levels during the entire testing period.

TOC

Attenuation of TOC was extremely poor after October 14, 1988, with a removal rate of less than 2%, and at times, achieving levels of outflow equal to those of the inflow. The latter results, obtained after October, contrast significantly with the earlier periods, when removal

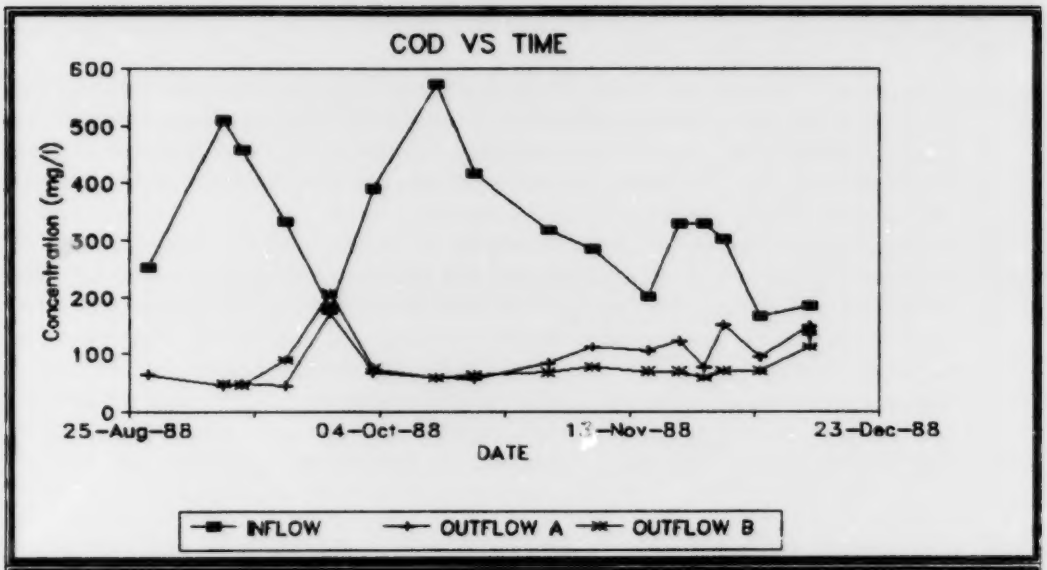

Figure 7. COD Variation over Time

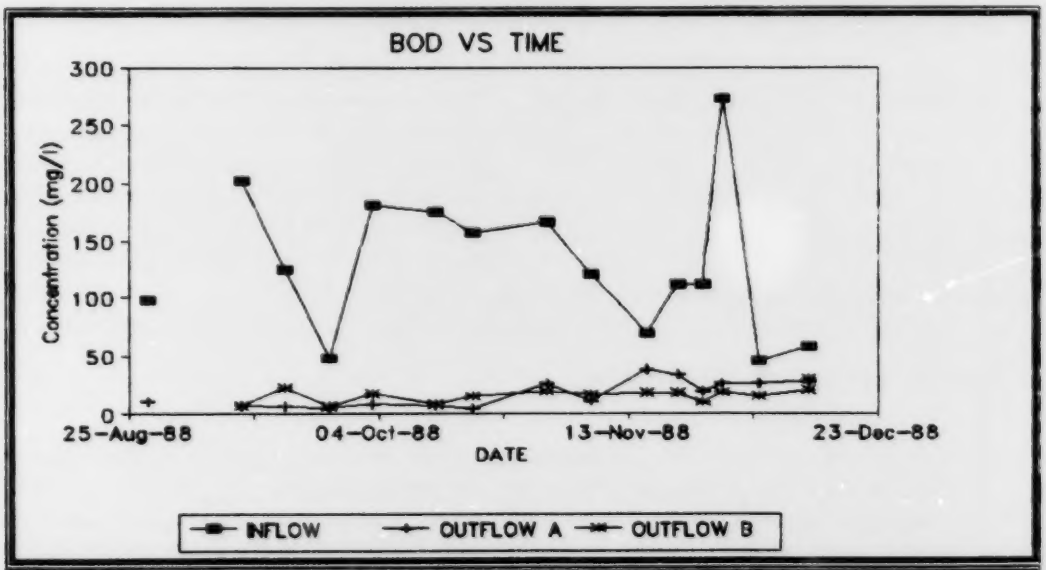

Figure 8. BOD Variation over Time

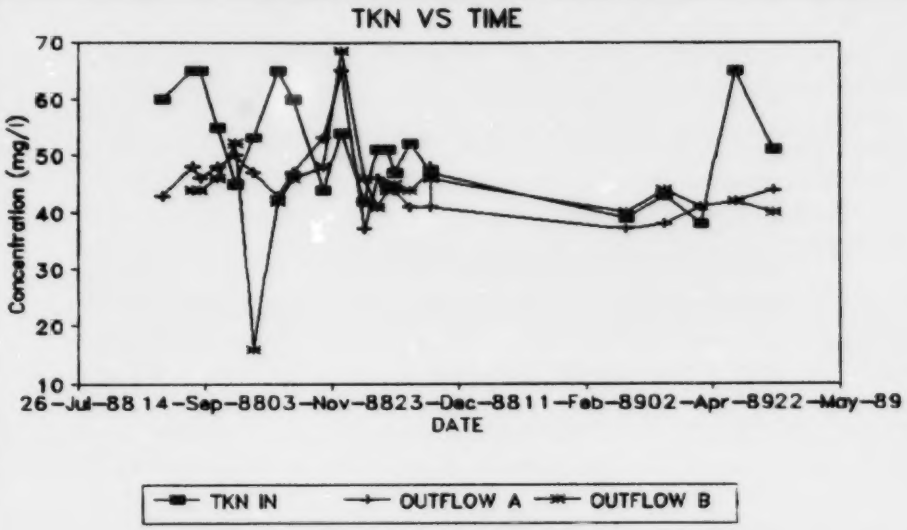

Figure 9. TKN Variation over Time

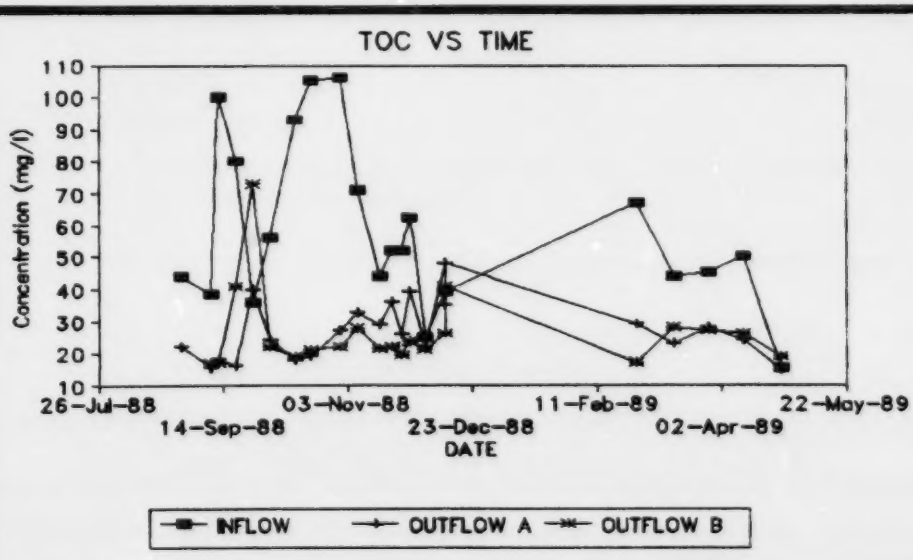

Figure 10. TOC Variation over Time

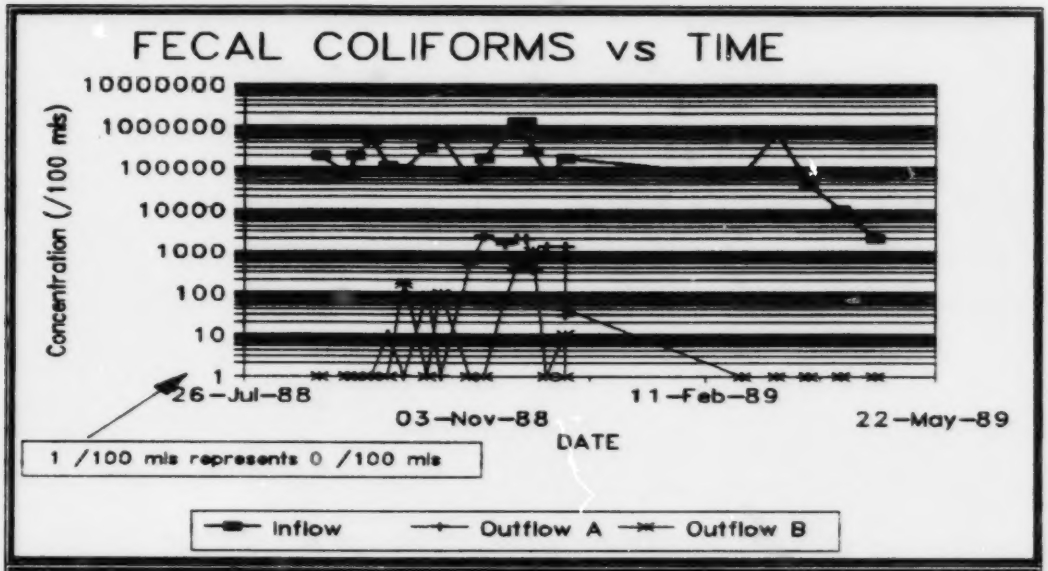

Figure 11. Fecal Coliforms Variation over Time

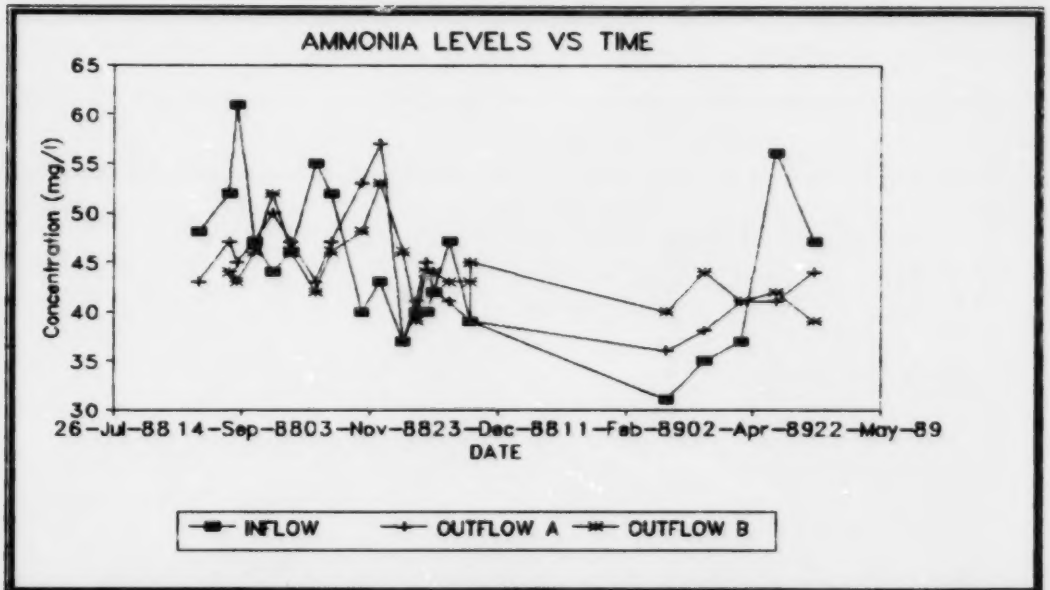

Figure 12. Ammonia Variation over Time

efficiency was of the order of 50 to 60 percent. A summary of the TOC treatment is found in figure 9. Fecal coliforms

Prior to October 3, 1988, no fecal coliforms were detected in the outflow of either column A or B (wt = -90 cm, transient inflow). Following this time, when the fixed head of effluent was established in the columns, fecal coliform levels began to increase to a maximum level of

2300 mpn per 100 ml in Column A and 1000 mpn per 100 ml in Column B. Though the coliform count was at times higher as flow rate increased, neither Column A nor B showed any type of consistent relationship between flow rate and fecal coliform removal. There does appear to be more substantial relationship between the relative position of the groundwater table below the clogging mat and fecal coliform mobilization. The results indicate that high coliform migration is possible when the water table is within 300 mm of the clogging mat. The bacterial movement was substantially diminished when the groundwater table was positioned 600 mm below the mat (the only exception to this was on December 12 which was believed to be caused by the sudden repositioning of the groundwater table).

Nitrogen

Throughout the experiment nitrogen removal was quite poor. Ammonia reduction varied over the entire analysis period with outflow levels of both columns at times exceeding inflow. Variation in ammonia levels appeared to be independent of either groundwater table location, or flow rate for both columns. Figure 12 summarizes the ammonia treatment of both columns over time. Comparison of figure 12 with that of figure 9, TKN, shows that most of the nitrogen entering and exiting the columns was in the form of ammonia and very little in the form of organic nitrogen. Nitrates & nitrites were not formed at all during the experimental period.

Discussion

Column Hydraulics

It is believed that the results of this experiment were influenced by several factors which made definitive conclusions very difficult. Principally, the effects of unsaturated flow, dynamic fluctuations in the clogging mat matrix, and fluctuating aerobic/anaerobic conditions all resulted in flow variations. The most dominant of these three influences was likely the fluctuations of the clogging mat matrix. Collimore et al[2] had recently noted a predictable trend in the clogging mat formation which led to a series of increases and decreases in the vertical flow of wastewater. Specifically, there were three phases of flow fluctuation which appeared as follows,

> " (A) A primary expansion where both the volume increased together with an increased resistance to flow,
>
> (B) The compression of the biomat which permitted greater interstitial flow and,
>
> (C) A secondary expansion of the biomat in which the volume of the biofilms increased while the rate of flow decreased. "

This hydraulic fluctuation is directly related to the dominant species occupying the biomat at a given period of time and the environment they impose upon the interstitial soil voids. It is presumed that some bacterial groups produced differing densities of colony and polysaccaride coats necessary to survive in the biomat.

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2 Observations on the Initial Phases of Biofouling by Iron Related Bacteria in Laboratory Simulated Water Well Microcosms, Colimore, R, Univ. of Regina.

The environment of the biomat was further modified by specific gases present at a given time. It is believed that as ponding continued to occur directly over the biomat region, very little oxygen was permitted to enter. Low oxygen would promote the development of an anaerobically dominant bacterial presence with a limited ability to treat certain elements of septic wastewater. As the bacterial colonies formed and died, wastewater flow rates would change, periodically allowing a complete depletion of ponded water above the clogging mat and the introduction of oxygen to the upper layers. However, over a prolonged period, the ponding of wastewater became continuous and under these conditions a purely anaerobic environment was likely established. A second possible phenomena caused by gas in the local biomat area was the effects of decreased water flow created by gas entrapment in the interstitial void region. It is a possibility that the formation of a perched water table could have forced any gases present in the biomat region to become traped, with the only passage available being vertically downward. This would likely be a manefestation of the experimental design caused mostly by the narrow 100 mm diameter tube containing the soil. It is not possible to prove or disprove the effects of gas entrapment on water movement, although future experiments should provide a means of dispersing gas from the area immediately underlying the clogging mat.

It is not possible at this time to define how unsaturated flow established itself within the columns owing to the strong influence of the clogging mat. It is evident that the characteristics of the clogging mat itself are dominating the vertical flow and require more detailed characterization in terms of a soil moisture-tension-conductivity inter-relationship.

Wastewater Treatment

As there was little opportunity for disolved oxygen to influence the wastewater in this column study, any organic nitrogen introduced above the clogging mat was either converted into ammonia or remained unprocessed after passing through the columns. This is readily observed in several instances where the outflow concentrations of ammonia exceeded the inflow values. As a result, NO_3 and NO_2 were not found to be produced at any time during the study. It is not certain to what degree nitrogen gas was produced, owing to the complexity of collecting and analyzing this substance, although little N_2 would be expected to form if the columns were normally operating under anaerobic conditions. The poor treatment of TOC, BOD and COD are likely a consequence of anaerobic conditions. Anaerobic bacteria are capable of some COD and BOD treatment but only to a limited extent, as evident in both columns.

The quality of the influent used to douse the experiment appeared to decline in concentration owing to the fact that the family had a new-born child which caused much higher initial TOC, COD and BOD loadings in the septic tank. As the child grew, the regularity of toilet loading declined, causing the relative concentrations to fall. It was interesting to note that dispite the decline in influent concentration, the outflows Oof TOC, BOD, COD remained at a relatively stable value.

Although conclusive results regarding the fate of bacteria are obscured by the design limitations of this experiment, it is highly probable that fecal coliforms are capable of migrating downward if the water table is within 300 mm of the biomat region. This finding is

consistent with results of Johnson and Atwater[3]. This bacterial movement probably stems from a tension saturated zone which provides a good aqueous pathway for bacteria to travel.

Conclusions

Inherent design deficiencies of the vertical column experiment make conclusions regarding vertical flow hydraulics unreliable. The dynamic nature of the clogging mat as it changes its microbial structure causes severe difficulties in obtaining useful information. The anaerobic conditions which likely exist in this experimental design do not necessarily reflect a true field condition, encouraging a biomat which contains a host of bacterial colonies unable to effectively treat wastewater. However, it is reasonable to conclude that there is a stronger possibility to expect bacterial groups to be transported away from the distribution trench if the groundwater is within 300 to 450 mm of the biomat.

Future experimental work involving the study of vertical flow hydraulics of clogging mats must consider the provision of a lateral gas exchange mechanism. It is also important to maintain very accurate records of the wastewater retention times above the clogging mat, as this will provide valuable insight into the various modes under which the passage of wastewater will oscillate. Knowledge of the short and long term variations of clogging mat hydraulics will ultimately dictate the design area of a distribution trench.

Implications

The most significant implication of this work is a need to provide adequate diffusion of gases to and from the clogging mat. If a mechanism for gas movement is not provided, a dominantly anaerobic bacterial biomat will develop which fails to effectively treat COD and BOD levels of septic wastewater, and develop extensive clogging of the host soil. Although deply constructed septic wastewater disposal trenches may not provide oxygen gas exchange into the clogging mat , shallower, near surface designs would. Furthermore, both deep and shallow designs will permit gasess to diffuse laterally away from the clogging mat and not inhibit wastewater flow. Area bed designs could possibly function in a manner similar to the columns, as the trench area (paricularily in the center) would not provide any pathways for oxygen exchange into the subsurface. This could explain why many older area beds installed in tight soils failed so readily. Design of any experimental system should also consider this problem of gas exchange to minimize the effects of developing biomat environments which differ substantially from a true in-situ contour trench condition (ie aerobic vs anaerobic).

A second implication of this work is that of the mobility of bacteria into the groundwater table should it rise to a height nearer than 300 to 450 mm from the biomat.

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3 Johnson and Atwater, "Role of Saturated and Unsaturated zone in Soil Disposal of Septic Tank Effluent, 1988, Can. J of Civil Eng, Vo 15, pp. 709-716.

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N.S. Department of Municipal Affairs

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N.S. Department of Industry Trade and Technology

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Johnson and Atwater, "Role of Saturated and Unsaturated zone in Soil Disposal of Septic Tank Effluent, 1988, Can. J of Civil Eng, Vo 15, pp. 709-716.

Freeze and Cherry, "Groundwater", Pentice Hall, 1979

APPENDIX A

Variation of Column Environment

# of Weeks		Phase Dates (Start of...)	Dousing Rate ml/sq.cm/day	Frequency times/day	Level * Below Top (mm)	Perched ** Water Table (mm)	Temp. (deg. C)	Influent	Measured Samples
	/\	13-May-88	1	3	1000	varies	2	S.T.E.	head
	|	24-May-88	1	3	800	varies	2	S.T.E.	head
22.00	|	30-Jun-88	1	3	300	varies	2	S.T.E.	head
	|	22-Jul-88	1	3	1000	varies	2	S.T.E.	head
	\/	30-Sep-88	1	3	600	varies	2	S.T.E.	head
	/\	14-Oct-88	varies	n/a	600	70	5	S.T.E.	Head+chem
	|	24-Oct-88	varies	n/a	300	70	5	S.T.E.	Head+chem
8.86	|	15-Nov-88	varies	n/a	0	70	5	S.T.E.	Head+chem
	|	30-Nov-88	varies	n/a	300	70	5	S.T.E.	Head+chem
	|	8-Dec-88	varies	n/a	600	70	5	S.T.E.	Head+chem
	\/	14-Dec-88	varies	n/a	1000	70	5	S.T.E.	Head+chem
4.43		15-Dec-88	varies	n/a	1000	70	5	lactos+STE	Head+chem
14.43	/\	15-Jan-89	0.6	3	1000	varies	22	S.T.E.	Head+chem
	\/	26-Apr-89	0.6	3	1000	varies	22	S.T.E.	Head+chem
49.71		27-Apr-90	end of project	-->-->	-->-->	-->-->	-->-->	-->-->	-->-->

	Notes
*	Distance below interface of gravel and sand (near top of column)
**	Distance above interface of gravel and sand (top of column)
STE-	Septic Tank Effluent
Head+chem-measured total/suction head, chemistry and bacteriology of inflow/outflow	

Appendix B

Piezometer Number

Date	Time	a1	a2	a3	a4	a5	a6	a7	a8	a9	a10	a11	a12	a13	a14	a15	Flow Rate	Piz'd Head	Water Table *
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	ml/hr	mm	mm
13-May-88		108.4	104.7	98.1	94.9	93.1	90.6	88.2	84.9	80.1	76.3	73.1	63.2	55.7	53.6	53.6	1cm/d	varies	1000
24-May-88		132.9	126.2	124.0	110.8	105.3	100.5	98.1	93.1	88.9	83.7	79.1	77.1	77.0	77.0	77.0	1cm/d	varies	1000
31-May-88		134.4	130.4	126.6	122.2	117.9	110.8	107.7	102.8	100.1	99.5	99.5	99.3	99.2	99.1	98.8	1cm/d	varies	800
10-Jun-88		137.8	133.5	123.6	118.8	113.9	111.2	105.6	99.9	94.0	92.7	90.9	90.6	91.4	89.3	88.9	1cm/d	varies	800
17-Jun-88		138.2	134.0	129.8	123.8	118.7	113.5	110.9	105.0	101.0	97.3	98.1	98.3	97.7	98.2	98.2	1cm/d	varies	800
20-Jun-88		139.7	135.7	131.3	125.4	120.2	114.9	111.7	105.6	100.0	96.0	95.5	93.1	94.0	93.0	91.4	1cm/d	varies	800
26-Jun-88		140.8	135.8	131.3	124.5	118.4	112.2	108.9	103.7	98.8	93.4	88.8	78.2	68.2	61.2	61.2	1cm/d	varies	800
30-Jun-88		143.5	136.9	132.3	126.3	125.1	124.5	124.5	123.4	123.3	122.2	122.2	121.8	121.7	121.5	121.5	1cm/d	varies	800
11-Jul-88		143.0	137.4	134.9	123.0	122.6	122.7	122.3	122.3	122.0	121.9	121.8	121.8	121.7	121.7	121.6	1cm/d	varies	300
11-Jul-88		142.1	136.3	131.1	124.5	122.8	122.4	122.4	122.3	122.3	122.0	121.9	121.8	121.8	121.8	121.6	1cm/d	varies	300
26-Jul-88	8:45 AM	140.8	135.8	131.3	124.9	119.2	112.8	109.1	104.1	98.9	93.3	88.7	78.0	67.9	61.0	61.0	1cm/d	varies	1000
26-Jul-88	10:15 AM	142.2	137.2	132.3	126.3	120.3	114.2	110.6	105.5	100.2	94.4	89.6	79.1	68.4	60.6	60.6	1cm/d	varies	1000
26-Jul-88	11:15 AM	142.1	137.1	132.3	126.4	120.4	114.3	111.0	105.6	100.4	94.7	89.6	79.3	68.5	60.6	60.6	1cm/d	varies	1000
26-Jul-88	1:35 PM	141.4	136.4	131.5	125.3	119.5	113.3	109.6	104.8	99.5	93.8	89.1	78.6	67.9	60.8	60.8	1cm/d	varies	1000
26-Jul-88	2:30 PM	142.3	137.1	132.1	126.3	120.1	113.9	110.6	105.4	100.2	94.4	89.4	79.0	68.2	60.8	60.8	1cm/d	varies	1000
4-Aug-88	10:00 AM	132.1	132.4	126.5	119.2	113.5	109.4	106.1	101.1	95.9	90.4	86.0	75.0	65.0	60.7	60.7	1cm/d	varies	1000
4-Aug-88	11:30 AM	137.3	132.3	126.9	119.4	113.7	108.6	105.7	101.5	96.3	90.8	86.5	75.5	65.3	60.7	60.7	1cm/d	varies	1000
4-Aug-88	12:35 PM	138.4	132.4	126.9	119.4	113.7	108.6	106.2	101.5	96.2	90.7	86.3	75.4	65.3	60.7	60.7	1cm/d	varies	1000
5-Aug-88	10:25 AM	141.5	132.8	126.8	119.3	113.6	108.6	106.2	101.3	96.2	90.6	86.3	75.3	65.2	60.7	60.7	1cm/d	varies	1000
5-Aug-88	2:35 AM	143.9	137.3	127.4	119.0	113.5	108.4	106.0	101.1	96.5	90.5	86.0	75.0	65.0	60.7	60.7	1cm/d	varies	1000
6-Aug-88	3:35 AM	143.9	136.6	128.4	119.0	113.2	108.0	105.9	101.0	95.8	90.3	85.9	75.0	65.0	60.9	60.9	1cm/d	varies	1000
6-Aug-88	11:55 AM	145.8	138.4	132.0	125.7	119.4	113.1	107.0	104.5	99.5	93.2	88.3	77.6	66.9	60.9	60.9	1cm/d	varies	1000
12-Aug-88		143.6	137.0	131.6	125.1	119.1	113.0	109.3	104.3	99.2	93.5	88.9	78.4	68.2	60.9	60.9	1cm/d	varies	1000
20-Aug-88		149.2	138.1	130.9	124.7	118.6	112.6	109.6	104.6	99.7	93.9	89.1	78.6	68.5	61.0	61.0	1cm/d	varies	1000
21-Sep-88		149.1	137.5	130.1	123.0	116.3	110.0	107.7	102.7	97.7	92.3	87.6	77.3	67.2	60.7	60.7	1cm/d	varies	1000
21-Sep-88	8:37 AM	148.8	138.0	130.2	122.8	116.5	111.1	107.8	103.5	98.3	92.8	88.4	77.9	67.8	60.5	60.5	1cm/d	varies	1000
21-Sep-88	9:00 AM	149.3	138.0	130.5	122.4	116.7	110.8	107.9	102.8	98.0	92.6	88.0	77.3	67.4	60.4	60.4	1cm/d	varies	1000
21-Sep-88	9:40 AM	149.5	138.4	131.1	122.7	117.0	111.0	108.3	102.9	98.1	93.0	88.0	77.2	67.4	60.8	60.8	1cm/d	varies	1000
21-Sep-88	10:40 AM	149.7	138.2	131.1	122.5	117.0	110.9	108.1	103.0	98.0	92.6	88.0	77.2	67.4	60.8	60.8	1cm/d	varies	1000
21-Sep-88	11:40 AM	149.4	137.5	130.6	122.3	116.5	110.7	107.8	102.8	97.8	92.3	87.8	77.2	68.2	60.8	60.8	1cm/d	varies	1000
21-Sep-88	1:40 PM	149.4	137.4	130.5	122.1	116.5	110.5	107.8	102.6	97.8	92.3	87.8	77.1	67.1	60.7	60.7	1cm/d	varies	1000
23-Sep-88	2:40 PM	149.0	137.0	129.7	122.0	116.2	110.3	106.9	102.5	100.5	92.0	87.6	77.0	67.1	60.4	60.4	1cm/d	varies	1000
30-Sep-88	4:40 PM	151.1	141.1	134.6	126.4	120.6	119.6	111.0	105.7	100.9	94.9	91.7	91.2	91.2	91.0	91.0	1cm/d	varies	700
13-Oct-88		148.4	138.1	132.6	124.6	118.6	112.5	109.3	104.2	99.3	94.0	91.2	91.1	91.1	91.1	91.0	19	varies	700
19-Oct-88		144.4	139.2	134.9	127.1	120.9	114.6	111.2	105.7	101.0	95.1	91.5	91.1	91.1	91.1	91.1	47	70	700
25-Oct-88		142.6	136.9	132.3	123.5	116.0	110.2	106.6	102.0	96.5	92.6	91.5	91.1	91.1	91.2	91.1	13	70	300
26-Oct-88		146.4	139.3	132.7	126.3	126.3	125.6	124.9	126.7	125.9	126.4	126.9	126.9	126.9	126.9	126.9		70	300
31-Oct-88		146.9	141.0	133.5	127.7	127.3	127.3	130.0	126.4	126.5	126.1	126.1	125.9	126.1	125.9	125.9	17	70	300
2-Nov-88		144.8	139.9	133.3	124.7	124.5	124.0	124.9	123.1	123.1	122.5	122.3	122.2	122.1	122.1	121.9	27	70	300
3-Nov-88		143.6	139.7	134.1	124.4	124.4	123.8	123.8	123.1	122.8	122.6	122.4	122.3	122.2	121.9	121.9	34	70	300
7-Nov-88		143.5	141.2	133.7	130.8	130.1	129.1	125.7	126.0	127.0	126.5	126.8	125.5	125.5	125.2	124.5	95	70	300
9-Nov-88		146.4	145.7	142.8	135.4	133.8	131.3	128.5	126.9	127.5	125.5	125.1	123.6	123.4	122.3	122.1	206	70	300
9-Nov-88	4:30	145.5	143.2	140.4	134.5	133.5	131.1	127.3	127.7	127.3	126.3	126.0	124.0	124.2	122.6	122.5	140	70	300
18-Nov-88	9:00	151.5	152.6	151.4	148.6	147.7	147.6	143.1	147.1	147.0	146.7	146.7	146.6	146.4	146.3	149.2	23	70	0
21-Nov-88	3:15	152.9	152.3	151.8	148.9	148.9	148.9	146.3	146.7	148.6	148.3	148.3	148.3	148.3	148.2	148.2	27	70	0
25-Nov-88	10:15	154.6	157.2	156.6	150.3	148.3	148.1	148.4	147.6	147.3	148.7	148.7	148.6	148.3	148.3	148.3	54	70	0
28-Nov-88	2:30	155.7	155.8	155.2	148.8	148.8	148.9	147.1	148.9	148.5	148.4	148.4	148.8	148.2	148.2	148.2	19	70	0
30-Nov-88	9:42	154.1	157.9	157.1	137.6	140.3	140.5	135.1	140.5	133.5	140.3	139.7	140.3	140.1	140.1	140.1	40	70	0
2-Dec-88	15	156.4	156.5	155.5	138.5	126.6	125.9	129.5	124.5	128.6	123.5	123.4	122.9	125.3	122.2	122.1	95	70	300
2-Dec-88	11	156.4	156.1	153.7	137.3	127.4	126.4	127.9	125.1	127.8	123.9	123.4	123.3	122.5	122.4	122.1	140	70	300
7-Dec-88	17.75	150.5	152.8	150.9	130.9	124.6	123.3	123.9	122.9	123.3	122.4	122.1	122.1	122.1	122.1	122.1	32	70	300
8-Dec-88	10	152.3	152.9	151.5	130.7	124.4	123.4	123.7	122.8	123.3	122.6	122.1	122.1	122.1	122.1	122.1	30	70	300
9-Dec-88	9.75	152.4	144.0	141.7	128.7	104.5	100.6	161.5	94.5	161.5	93.5	97.5	92.2	121.8	91.3	91.1	120	70	600
9-Dec-88	10.25	151.8	145.8	143.6	125.8	104.7	99.9	161.5	94.5	161.5	93.1	161.5	92.3	91.7	91.3	91.1	125	70	600
9-Dec-88	14.8	152.2	146.8	144.0	128.1	105.3	100.1	96.9	94.5	93.7	93.2	92.6	92.3	91.8	91.3	91.1	134	70	600

* Distance Relative To Bottom of Clogging Mat

Piezometer Number

Date	Time	a1	a2	a3	a4	a5	a6	a7	a8	a9	a10	a11	a12	a13	a14	a15		Flow Rate	Pz'd Head	Water Table *
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm		ml/hr	mm	mm
12-Dec-88	16:25	152.4	147.1	144.4	123.1	110.5	104.3	98.2	94.3	123.2	82.1	82.0	91.6	91.3	91.2	91.1		75	70	600
12-Dec-88	10:00	152.4	147.1	144.7	123.1	110.4	104.3	98.2	94.3	83.2	122.2	122.0	121.6	91.3	91.2	91.1		78	70	600
13-Dec-88	12:00	152.9	148.3	146.5	122.5	109.8	103.9	98.0	93.9	82.8	82.2	82.0	91.6	91.3	91.2	91.1		71	70	600
14-Dec-88	13:17	152.6	144.1	139.2	121.7	108.8	104.1	98.0	88.1	84.1	80.9	77.0	71.9	71.6	72.3	71.5		193	70	600
15-Dec-88	11:40	151.9	142.5	138.5	121.1	107.2	102.5	92.5	84.3	81.3	78.6	71.4	63.5	62.6	61.5	61.2		275	70	1000
20-Jan-89	3:23	151.0	150.1	150.0	139.7	133.0	128.8	123.3	117.8	113.5	107.5	102.5	90.5	82.0	71.9	71.5		13	35	1000
27-Feb-89	10:45	145.6	140.7	138.9	132.4	120.1	117.9	115.3	109.5	105.0	98.8	94.2	88.6	72.8	70.4	70.0		6	40	1000
7-Mar-89	10:5	143.5	141.0	138.0	125.3	113.7	111.3	107.4	104.0	97.5	91.5	87.2	84.2	70.3	69.8	69.7		3	50	1000
3-Apr-89	10:27	146.4	144.1	143.3	136.2	126.3	122.7	119.4	113.4	109.9	102.9	98.3	90.5	78.2	70.2	70.2		10	70	1000
7-Apr-89	10:56	149.4	145.3	144.5	137.2	128.7	123.4	120.4	116.0	110.6	103.9	99.3	89.5	77.5	80.4	70.2		8	50	1000
11-Apr-89	5:15	149.3	146.5	144.7	138.6	128.1	123.0	119.8	114.3	110.1	103.4	98.8	91.7	77.2	70.2	70.1		7	80	1000
18-Apr-89		150.5	148.1	145.4	138.1	130.8	124.7	121.2	113.7	111.5	103.7	101.1	87.7	79.5	70.2	70.2		6	70	1000
28-Apr-89		70.4	147.3	145.1	138.4	129.2	122.5	119.9	114.3	110.0	103.5	99.3	88.3	77.5	70.3	70.2		7	80	1000

* Distance Relative To Bottom of Cleaning Nut

Date	Time	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10	B11	B12	B13	B14	B15	Flow Rate	Fix'd Head	Water Table *
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	ml/hr	mm	mm
13-May-88		94.0	90.6	88.7	86.3	85.9	84.9	76.5	76.1	94.2	88.9	84.1	55.6	50.1	48.7	48.6	1cm/d	varies	1000
24-May-88		135.2	128.4	122.1	113.9	107.3	98.8	95.8	90.9	83.5	81.8	76.8	67.2	65.4	65.2	65.0	1cm/d	varies	1000
31-May-88		137.1	131.9	127.5	122.3	113.7	110.2	104.3	97.3	87.1	98.2	98.2	98.1	98.1	97.4	95.9	1cm/d	varies	800
10-Jun-88		142.3	134.7	128.6	124.7	120.2	113.3	108.9	102.4	98.5	93.2	93.2	93.0	92.7	92.3	91.4	1cm/d	varies	800
17-Jun-88		141.8	136.8	130.6	126.1	119.7	112.3	107.1	101.0	98.9	92.8	92.4	92.3	92.2	92.2	91.8	1cm/d	varies	800
20-Jun-88		142.5	136.3	131.1	125.4	118.4	110.9	103.9	98.4	98.1	92.4	92.0	91.9	92.0	91.9	91.5	1cm/d	varies	800
20-Jun-88		142.5	136.3	131.1	125.4	118.4	110.9	103.9	98.4	98.1	92.4	92.0	91.9	92.0	91.9	91.5	1cm/d	varies	600
30-Jun-88		144.0	138.4	131.0	126.9	121.4	122.4	122.4	122.3	110.5	121.8	121.7	121.6	121.5	121.4	121.1	1cm/d	varies	600
11-Jul-88		148.9	137.5	133.4	127.0	121.8	121.7	121.7	121.6	116.1	121.4	121.4	121.3	121.3	121.3	121.2	1cm/d	varies	300
11-Jul-88		148.0	137.3	132.5	128.1	121.7	121.6	121.5	121.5	116.1	121.4	121.4	121.2	121.4	121.5	121.0	1cm/d	varies	300
28-Jul-88	8:45 AM	144.7	137.3	131.9	126.0	119.3	111.2	105.3	100.3	98.2	88.7	83.0	71.2	62.0	59.8	59.7	1cm/d	varies	1000
28-Jul-88	10:15 AM	144.5	137.0	131.7	125.8	119.3	111.0	105.2	100.2	98.2	88.7	82.8	71.1	62.0	59.7	59.7	1cm/d	varies	1000
28-Jul-88	11:15 AM	145.2	138.5	132.8	126.9	119.3	111.9	105.6	100.3	98.0	88.8	83.9	71.8	62.2	59.9	59.8	1cm/d	varies	1000
28-Jul-88	1:35 PM	145.0	138.2	132.8	127.0	119.4	112.6	106.8	100.5	98.1	89.0	85.3	72.5	62.5	60.1	60.0	1cm/d	varies	1000
28-Jul-88	2:30 PM	144.8	138.2	132.7	127.0	119.4	112.6	106.9	100.5	97.9	89.0	84.9	73.1	62.8	60.2	59.6	1cm/d	varies	1000
28-Jul-88		144.8	137.5	132.1	126.2	118.9	111.3	105.0	99.9	97.8	88.4	83.1	71.2	62.0	59.8	59.7	1cm/d	varies	1000
4-Aug-88	10:00 AM	140.7	133.9	127.6	122.5	120.0	109.2	108.0	101.9	98.3	90.4	84.3	74.4	72.5	72.9	72.3	1cm/d	varies	1000
4-Aug-88	11:30 AM	142.1	134.0	127.8	122.3	119.8	109.1	105.8	101.7	99.1	90.2	84.3	74.3	72.5	72.8	72.5	1cm/d	varies	1000
4-Aug-88	12:35 PM	145.8	134.0	127.6	122.1	119.5	108.9	105.5	101.4	98.0	90.0	84.1	74.3	72.4	73.0	72.4	1cm/d	varies	1000
5-Aug-88	10:25 AM	145.9	138.9	127.5	122.0	119.3	108.8	105.5	101.3	98.0	89.9	84.0	74.2	72.4	72.5	72.4	1cm/d	varies	1000
5-Aug-88	2:35 AM	145.6	139.0	128.9	122.1	119.1	108.6	105.3	101.1	97.9	89.7	83.7	74.2	72.5	72.6	72.5	1cm/d	varies	1000
5-Aug-88	3:35 AM	145.4	139.2	129.9	122.5	118.9	108.5	105.1	100.9	97.9	89.6	83.5	74.2	72.5	72.6	72.5	1cm/d	varies	1000
5-Aug-88	11:55 AM	148.7	142.1	134.2	128.4	120.1	115.2	109.4	102.7	97.4	93.6	89.7	78.4	75.2	75.6	74.7	1cm/d	varies	1000
12-Aug-88		145.4	138.7	131.2	125.4	120.6	112.7	107.9	102.1	97.7	94.6	88.9	75.5	63.8	53.6	53.4	1cm/d	varies	1000
26-Aug-88		149.5	146.9	138.6	133.0	125.8	120.4	115.6	110.1	101.6	99.9	94.8	84.1	72.9	63.0	60.9	1cm/d	varies	1000
21-Sep-88	8:37 AM	139.5	137.8	132.5	126.0	121.6	112.5	110.0	114.9	101.8	93.4	87.6	74.1	65.5	62.5	62.5	1cm/d	varies	1000
21-Sep-88	9:00 AM	141.9	137.7	134.3	127.9	121.1	115.0	109.6	104.5	101.5	93.5	89.3	78.5	67.8	61.2	60.3	1cm/d	varies	1000
21-Sep-88	9:40 AM	142.2	138.4	134.2	128.2	120.8	115.3	110.3	105.0	102.0	94.5	90.0	78.8	68.0	60.9	60.1	1cm/d	varies	1000
21-Sep-88	10:40 AM	141.5	138.4	133.8	127.7	121.8	114.5	110.2	104.9	101.9	94.8	89.6	78.7	67.5	61.0	60.4	1cm/d	varies	1000
21-Sep-88	11:40 AM	140.8	138.0	132.6	128.8	121.7	113.7	110.1	104.9	101.9	94.7	89.0	78.3	68.9	60.8	60.2	1cm/d	varies	1000
21-Sep-88	1:40 PM	138.3	138.2	130.2	124.3	120.0	110.8	108.8	103.8	101.7	92.7	88.4	76.5	64.6	60.9	60.3	1cm/d	varies	1000
21-Sep-88	2:40 PM	137.1	135.2	129.0	123.0	119.0	109.5	107.6	102.6	101.7	91.4	85.1	74.4	63.6	60.9	60.3	1cm/d	varies	1000
23-Sep-88	4:40 PM	139.6	134.9	123.0	117.2	110.5	108.5	101.2	101.0	91.1	88.9	78.3	65.2	60.5	60.2	60.2	1cm/d	varies	1000
30-Sep-88		145.1	144.2	133.8	127.1	121.1	114.4	109.8	103.9	98.8	94.8	92.0	92.3	92.3	91.8	91.8	1cm/d	varies	700
13-Oct-88		141.2	141.7	136.0	128.4	122.1	115.4	111.4	105.8	99.2	98.2	93.9	93.5	93.1	92.9	92.8	26	varies	700
19-Oct-88		142.1	136.4	122.9	125.8	119.6	112.4	108.0	112.1	97.7	93.9	94.1	93.8	93.8	93.9	93.8	10	70	700
25-Oct-88		144.2	138.8	134.9	127.5	120.4	114.8	109.8	103.4	97.5	94.3	94.3	94.2	94.3	94.3	94.3	11	70	300
25-Oct-88		144.2	138.8	134.9	127.5	120.4	114.8	109.8	103.4	97.5	94.3	94.3	94.3	94.3	94.3	94.3	11	70	300
28-Oct-88		142.9	137.9	133.0	126.1	121.1	122.2	122.2	122.2	105.0	122.2	122.2	122.2	122.2	122.2	122.2		70	300
31-Oct-88		142.3	137.9	132.5	125.9	122.3	123.3	122.3	122.3	122.3	122.3	122.3	122.3	122.3	122.3	122.3	7	70	300
2-Nov-88		149.5	148.0	140.2	133.1	124.1	123.3	123.3	123.3	115.5	122.8	122.8	122.8	122.7	122.7	122.5	14	70	300
3-Nov-88		148.1	146.5	138.5	131.5	124.4	123.2	123.3	123.3	116.4	122.8	122.8	122.6	122.5	122.5	122.3	16	70	300
7-Nov-88	4:30	150.3	149.7	146.3	139.8	130.2	125.7	125.3	124.8	118.7	124.3	124.1	123.7	123.3	122.6	122.3	65	70	300
9-Nov-88	9:00	145.0	142.8	138.0	130.5	128.1	124.5	124.3	124.1	119.7	123.6	123.5	123.3	123.2	122.6	122.3	41	70	300
9-Nov-88	3:15	147.6	147.2	140.5	135.0	130.3	124.9	124.5	124.2	119.4	123.7	123.7	123.6	123.3	122.9	122.3	57	70	300
16-Nov-88	10:15	152.2	151.2	149.5	149.1	146.8	146.3	146.3	146.1	130.3	147.9	147.8	147.8	147.4	147.3	147.0	26	70	0
21-Nov-88	2:30	150.2	149.7	148.5	148.3	148.1	148.0	147.7	147.7	158.5	147.4	147.3	147.3	147.2	147.0	146.9	30	70	0
21-Nov-88	9:42	150.2	149.9	148.5	148.3	148.1	148.0	147.7	147.7	147.5	147.4	147.3	147.3	147.2	147.0	146.9	26	70	0
23-Nov-88	15	151.4	150.7	149.8	149.5	149.4	149.4	148.8	148.8	148.7	148.7	148.4	148.4	148.2	148.1	147.8	71	70	0
25-Nov-88	11	151.4	150.7	149.8	149.5	149.4	149.4	148.8	148.8	148.7	148.7	148.3	148.4	148.2	148.1	147.8	26	70	0
28-Nov-88	17.75	148.3	148.3	148.0	148.3	148.0	148.4	148.0	148.1	148.1	148.0	147.9	147.9	147.9	147.9	147.9	9	70	0
30-Nov-88	10	153.2	151.4	143.1	136.5	128.8	122.7	122.5	122.5	122.5	122.5	122.3	122.2	122.2	122.0	121.8	28	70	0
2-Dec-88	9.75	152.7	151.0	142.4	135.8	128.9	122.7	122.3	122.3	122.3	122.3	122.3	122.1	122.1	122.0	122.0	2	70	300
2-Dec-88	10.25	152.7	151.0	142.5	135.4	129.3	122.7	122.3	122.3	122.3	122.3	122.3	122.1	122.1	121.9	121.9	90	70	300
2-Dec-88	14.8	148.8	143.9	138.3	131.1	127.3	122.1	122.2	122.2	122.2	122.3	122.2	121.9	121.9	121.9	121.9	12	70	300
7-Dec-88	16.25	149.2	144.7	139.2	132.5	127.9	122.7	122.3	122.0	122.4	122.5	122.3	122.1	122.1	122.1	122.1	17	70	300
9-Dec-88	10:00	141.1	136.5	129.5	123.1	119.1	111.9	106.6	99.4	95.5	93.5	93.3	93.0	92.6	92.6	92.4	41	70	600

* Distance Relative To Bottom of Clogging Mat

Date	Time	B1	B2	B3	B4	B5	B6	B7	B8	B9	B10	B11	B12	B13	B14	B15	Flow Rate	Fix'd Head	Water Table *
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	ml/hr	mm	mm
9-Dec-88	12:00	148.3	141.0	134.2	132.5	120.5	119.4	115.6	105.5	102.8	98.5	93.5	93.1	122.7	92.5	92.0	51	70	600
9-Dec-88	13:17	147.4	142.5	135.6	132.5	121.1	119.3	114.5	104.9	103.3	97.3	93.5	93.0	92.7	92.7	92.0	50	70	600
12-Dec-88	11:40	143.2	141.0	131.5	127.1	122.8	113.9	109.5	103.9	98.5	94.2	93.0	92.8	92.5	92.4	92.3	38	70	600
12-Dec-88	3:23	143.2	140.9	134.5	127.2	122.9	113.8	109.6	103.9	98.5	94.2	93.0	92.8	92.5	92.4	92.3	33	70	600
13-Dec-88	10:45	141.2	137.3	132.8	123.8	120.7	110.0	108.3	102.1	95.6	93.1	93.1	92.6	92.5	92.3	92.3	27	70	600
14-Dec-88	10:5	137.5	133.1	127.4	114.8	110.4	101.4	97.3	91.6	88.2	80.8	74.5	65.3	65.0	64.5	64.5	81	70	600
16-Dec-88	10:27	134.9	131.4	124.3	116.4	109.9	102.1	97.7	90.7	87.0	81.7	76.8	63.8	63.3	62.5	62.5	100	70	1000
20-Jan-89	10:56	135.1	132.1	124.7	119.1	111.8	104.9	100.1	92.3	89.1	82.5	77.5	70.4	70.2	70.0	69.6	13	70	1000
27-Feb-89	5:15	140.3	141.5	133.6	127.2	119.6	111.6	107.3	99.9	93.9	89.6	84.6	73.4	70.4	70.3	69.4	3	40	1000
7-Mar-89		144.3	139.5	131.7	123.6	117.4	108.7	104.5	98.0	93.0	87.5	80.7	71.1	69.5	69.5	69.5	3	50	1000
3-Apr-89		141.8	135.5	129.5	118.5	114.8	103.8	101.3	95.7	89.1	84.9	77.8	69.9	69.8	69.7	72.5	4	60	1000
11-Apr-89		142.8	137.0	128.9	122.3	114.6	107.2	102.7	95.4	91.3	85.1	80.3	71.2	69.8	70.5	69.3	1	60	1000
18-Apr-89		146.9	141.1	133.6	126.0	119.1	111.0	108.6	99.8	95.3	89.7	84.4	73.5	70.5	69.8	70.5	2	80	1000
26-Apr-89		144.9	138.5	131.5	123.0	116.5	108.4	104.1	97.7	92.6	87.4	82.1	71.4	69.4	69.4	69.4	1	100	1000

* Distance Relative To Bottom of Clogging Mat

Appendix C

VERTICAL COLUMN A & B
BACTERIAL & CHEMICAL INFLOW AND OUTFLOW

	TKN			AMMONIA			TOC			COD			BOD			FECAL COLIFORMS			FLOW- RATE		Depth to	Phase
DATE	Inflow	AOut	B Out	Inflow	AOut	B Out	Inflow	AOut	B Out	Inflow	AOut	B Out	Inflow	AOut	B Out	Inflow	AOut	B Out	A	B	Water-	
	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	mg/l	/100 mls	/100 mls		ml/hr	ml/hr	Table (cms)	
28-Aug-88	60.0	43.0		48.0	43.0		44.0	22.0		252.0	64.0		98.0	9.8		200000	0	0			100.0	
9-Sep-88	65.0	48.0	44.0	52.0	47.0	44.0	38.5	15.7	16.8	510.0	46.0	49.0				60000	0	0			100.0	I
12-Sep-88	65.0	46.0	44.0	61.0	45.0	43.0	100.0	18.0	17.0	457.6	49.3	47.0	202.0	6.4	6.7	200000	0	0			100.0	
19-Sep-88	55.0	48.0	46.0	47.0	47.0	46.0	60.0	16.0	41.0	332.8	45.3	66.4	126.0	6.2	22.0	450000	0	0			100.0	
26-Sep-88	45.0	50.0	52.0	44.0	50.0	52.0	36.0	40.0	73.0	182.0	171.6	208.0	48.0	4.6	5.9	110000	10	0			100.0	
3-Oct-88	53.0	47.0	16.0	46.0	47.0	46.0	56.0	24.0	22.0	390.0	68.9	75.0	181.0	8.3	17.0	90000	0	160			60.0	
13-Oct-88	65.0	43.0	42.0	55.0	43.0	42.0	93.0	17.9	18.9	572.0	60.0	59.4	174.0	6.7	8.1	300000	80	0	26.0	19.0	60.0	
19-Oct-88	60.0	47.0	46.0	52.0	47.0	46.0	105.0	19.0	21.0	416.0	57.0	63.0	157.0	4.3	15.0	560000	0	90	11.0	47.0	60.0	
31-Oct-88	44.0	53.0	48.0	40.0	53.0	48.0	106.0	27.0	22.0	319.0	84.0	68.4	166.0	26.0	18.6	50000	480	0	13.7	27.3	30.0	II
7-Nov-88	64.0	65.0	68.4	43.0	57.0	53.0	71.0	32.7	27.6	385.0	112.0	77.4	122.0	10.8	16.4	170000	2300	0	56.6	140.0	30.0	
16-Nov-88	42.0	37.0	46.0	37.0	37.0	46.0	44.0	28.8	21.4	201.6	106.0	69.6	70.0	38.0	17.7	700000	1500	70	36.0	36.0	0.0	
21-Nov-88	51.0	46.0	41.0	40.0	41.0	39.0	52.0	36.0	22.0	330.4	123.2	70.2	112.0	33.6	17.9	1200000	2000	400	26.0	54.0	0.0	
25-Nov-88	51.0	45.0	44.0	40.0	45.0	44.0	52.0	25.8	19.2	330.4	78.0	60.0	112.0	19.0	9.9	1200000			54.0	54.0	0.0	
28-Nov-88	47.0	44.0	45.0	42.0	42.0	44.0	62.5	39.3	23.6	301.6	151.2	71.0	273.0	25.6	18.2	240000	350	1000	28.0	40.0	26.0	
4-Dec-88	52.0	41.0	44.0	47.0	41.0	43.0	25.0	22.0	21.0	166.4	93.6	70.6	45.6	26.0	14.9	60000	1300	0	17.0	32.0	30.0	
12-Dec-88	47.0	41.0	47.0	39.0	39.0	43.0	39.0	48.0	26.0	184.8	150.8	112.0	58.7	28.0	20.0	170000	1300	10	38.0	74.0	60.0	
12-Dec-88	47.0	48.0	46.0	39.0	45.0	45.0	39.0	35.0	41.0	184.8	134.4	140.0	58.7	27.0	30.0	170000	30	10	28.0	75.0	60.0	
12-Dec-88	47.0	41.0	46.0	39.0	39.0	45.0	39.0	48.0	41.0	184.8	150.8	140.0	58.7	28.0	30.0	170000	40	0	33.0	77.5	60.0	
27-Feb-89	39.0	37.0	40.0	31.0	36.0	40.0	67.0	29.0	17.0							60000	0	0	3.0	6.0	100.0	
14-Mar-89	43.0	38.0	44.0	35.0	38.0	44.0	44.0	23.0	28.0							700000	0	0			100.0	
28-Mar-89	38.0	41.0	41.0	37.0	41.0	41.0	45.0	28.0	27.0							40000	0	0	6.2	8.4	100.0	III
11-Apr-89	65.0	42.0	42.0	56.0	41.0	42.0	50.0	24.0	26.0							10000	0	0	1.4	7.3	100.0	
26-Apr-89	51.0	44.0	40.0	47.0	44.0	39.0	15.4	15.0	19.0							3000	0	0	4.0	21.0	100.0	

Note: 2/3 gal/sq.ft/day=5ml/sq.cm/hr or 5 cm /hr

Field Evaluation of In-Situ Permeability Tests For Site Characterization and Septic Field Design

G.M. Charles, D.H. Waller

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Introduction

The proper design of a septic wastewater disposal field for any lot relies greatly upon an accurate assessment of the soil. The most important characteristic of the soil is its ability to pass effluent at a rate which will allow adequate treatment and avoid ponding. The rate at which the soil will pass effluent is expressed in terms of soil permeability. Technical guidelines published by the Nova Scotia Department of Health and Fitness (N.S.DoH) rely on a specific range of soil permeability for the design of on-site systems. Therefore, a method of accurately measuring soil permeability is required.

In the past, percolation tests have been the most common method of estimating permeability of a soil. Unfortunately, this test has been found to be influenced by several variables, including:

- the radius of the test hole
- the depth of water in the test hole
- the field saturated conductivity of the soil
- the initial pore water pressure
- proximity of the local water table to the hole
- impermeable subsoil strata below the hole

A more common method used at present involves a visual inspection of the soil horizons and classification of the soil type. While these two methods are still often used, there is a need to adopt a method of in-situ evaluation for use in difficult design situations.

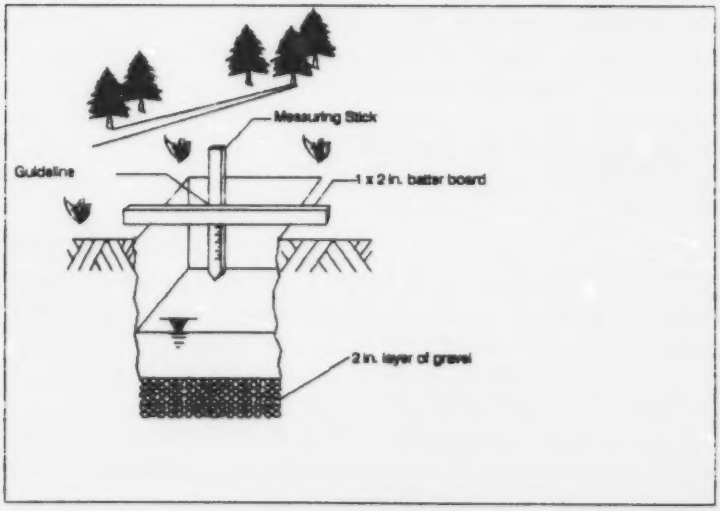

Figure 1. The Standard Percolation Test

Objectives

To evaluate and compare appropriate in-situ permeability testing methods for use in designing on-site sewage disposal fields in Nova Scotia.

Method of Investigation

The development of simple, reliable tests for in-situ permeability evaluation was the primary goal of this study. Seven different methods of permeability assessment were considered for evaluation and included: the Constant head permeameter, the Double Ring Infiltrometer, the shallow well pump-in test, a chloride tracer test, the standard percolation test, the A.S.T.M. falling head test, and a Soil Classification Method. The latter three of this list were subsequently omitted from the investigation for the following reasons.

1. The percolation test was proven by previous researchers to be too unpredictable, being influenced by several uncontrolable site factors as previously discussed.

2. The visual inspection method was too arbitrary in nature and lacked a method of giving exact numerical data.

3. The falling head test, although attempted under phase 2 of this study (Sackville site), failed to give reasonable results due to "short circulting" caused by wormholes and roots found in the soil matrix of the upper horizons.

The study was conducted in two phases. The first phase was an attempt to test several different sites in the Halifax county region with each of the different apparatus. It should be noted that the chloride tracer test was only performed on the second site of phase 2. In the first phase, each of the first three methods were used on six separate sites. Phase two was a progressive step evolving from phase one and arose from the erratic and inconclusive data which was obtained. For the second phase, two sites were chosen and each of the three instruments were tested a minimum of fifteen times. Site 1 was located in Lower Sackville and site 2 was in Port Maitland, N.S.

Description of Sites

The six lots of phase 1 and the first lot of phase two were located in the Halifax county region and lot 2 of phase 2 was located in Yarmouth County, 17 km north of the town of Yarmouth. The typical Halifax county soil type as assessed in this project is reddish brown, bimodal clast fabric, with clay dominated by kaolinite and containing varying amounts of pebbles, cobbles and small boulders.

Figure 2, Areas included in this Study

Soils of the lots tested under phase 1 of this study were somewhat varied, ranging from a silty sand to a silty, sandy clay. As a result, little comparison can be made between each of the instruments from one lot to the next.

The Sackville site of phase 2 was a sandy clay. The sieve analysis of this lot can be found in Appendix A. Although over 50% of this soil passes the 200 sieve, the nature of this

structured glacial till supports a reasonably high permeability. The Port Maitland soil is somewhat similar to the Sackville site, consisting of a high fines content till.

Figure 2. Areas included in Study

Equipment and Methods

Constant Head Permeameter

Details of the development and theory underlying this apparatus may be found in references 1 and 3. The constant head permeamter is an auger hole method which maintains a constant head within a small diameter dug well and monitoring the rate of change of inflow over a given time period. The constant head permeamter is illustrated in figure 3.

As shown, the constant head permeameter is a simple 75 mm diameter upper reservoir mounted on a 40 mm diameter standpipe. Readings are taken directly from the upper reservoir and the height, h, is established by raising the air inlet tube.

The test procedure is as follows:

1. A 75mm diameter hole is augured to a varying depth. In the case of soils containing a high fines content, a 50 mm diameter coarse bristle or wire brush (a wire brush was used in this study) is run up and down the inside of the hole to remove any effects of smearing. If the soil is particularly unstable, some coarse sand or pea gravel can be placed in the hole to a depth of 50mm consideration must be given to the height of pea stone when setting the water levels of the instrument and using this value "h" in the governing equation (see below)).

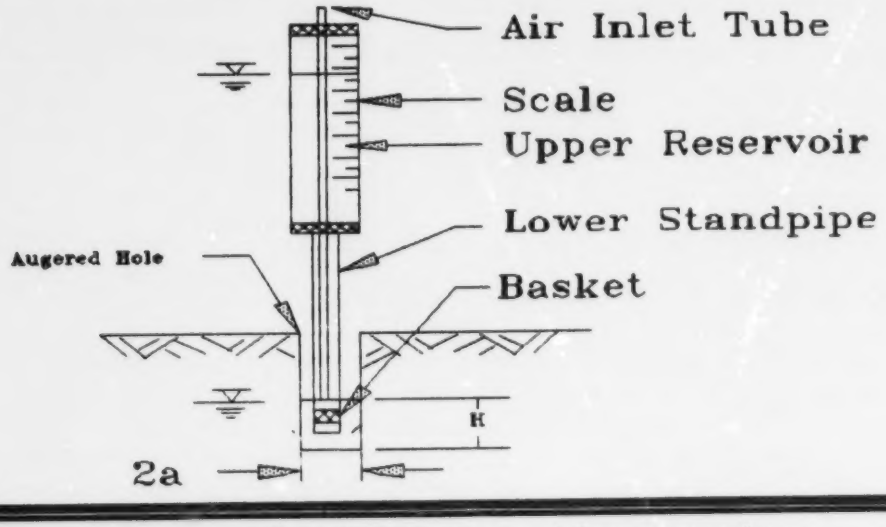

Figure 3. The Constant Head Permeameter

2. The permeameter is then placed inside the hole with the upper reservoir filled with water (It is important to let the hole fill with water from the instrument and not by pre-filling, as unusual values can result. See reference 5 on the effects of vacuuming in the upper reservoir). If pea stone is used, it may be placed around the sides of the instrument prior to filling the hole with water.

3. The variation of inflow rate is noted by recording the scale reading of the upper reservoir every minute (or every five minutes in the case of soils with high fine contents).

4. Testing may be discontinued when the rate of descent of the upper reservoir is constant over five successive readings.

5. If required, the permeameter may be removed and the core depth extended to allow the testing of deeper soil horizons. Steps 1 through 4 are then repeated.

The permeability may then be calculated using equations 1 and 2

$$C_1 = \frac{C \cdot A}{2\pi H^2 \left[1 + \frac{C}{2}\left(\frac{a}{H}\right)^2\right]}$$

Equaton 1

$$K_{fs} = R\ C_1$$

Equaton 2

Where,

R = Infiltration rate (m/s)
K_{fs}=field saturated conductivity (m/s)
C_1=constant
C= Constant derived from figure 4
A= radius of the upper reservoir (cm^2)
a= radius of the auger hole (cm)
H= height of water inside the hole (cm)

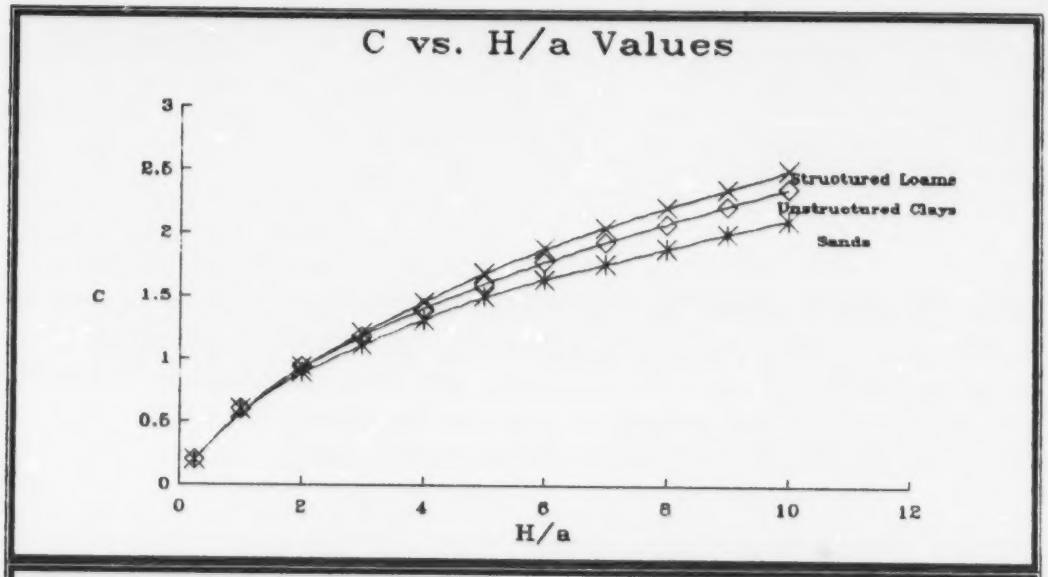

Figure 4. Constants used in the Guelph Permeameter Calc'n

Double Ring Infiltrometer

The standard A.S.T.M. method for Double Ring Infiltrometers was modified slightly to achieve a smaller more lightweight instrument. This apparatus uses two steel rings, one 300mm in diameter and the other 200 mm in diameter. A constant head is maintained within the inner ring using a mariotte tube. The same head is maintained in the annular space between the two rings using a simple external reservoir.

The apparatus and theoretical flow fields are illustrated in figure 5. The basic principal of this device is to create a vertical downward flow in the soil. An outer ring is used to maintain a region of divergent flow, while an inner ring is intended to sustain a vertical flow which has a hydraulic gradient of one (providing the equipment is allowed to run for some time).

The procedure for this experiment is as follows

1) a 600 mm diameter hole is dug to remove the upper "L-H" horizon of topsoil, exposing a level, undisturbed "A" horizon below.

2) With the aid of a wire brush, the soil is scarified slightly to remove any smearing which may have resulted from the shovel blade.

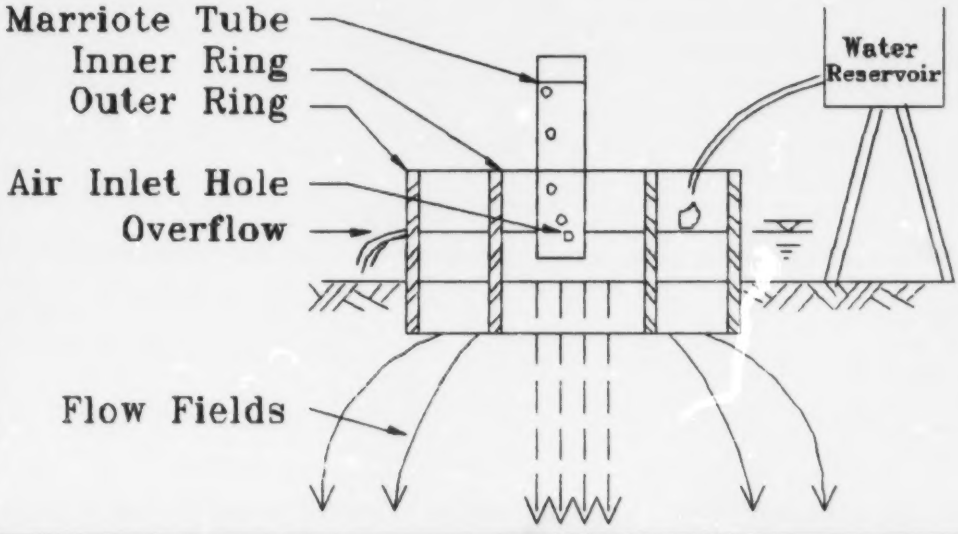

Figure 5. Constants used in the Guelph Permeameter Calc'n

3) The outer ring is place in the hole and driven down at least 100 mm with the aid of a maul and metal tamping plate.

4) The inner ring is then position symmetrically inside the outer ring and driven down so both rings are at the same depth.

5) Water is fed to the outer ring using a constant feed reservoir while the inner ring is fed with a constant head "marriotte tube". During start-up, both rings are prefilled to minimize the loss of water particularly in the marriotte tube. The apparatus is designed so that the water level is identical for the inner and out rings.

6) The equipment is run for approximately one hour and periodic measurements are taken. Appendix B shows the data sheet used in maintaining a site record. An approximation of the soil porosity (n) is used, as the final result is not too sensitive to marginal variations ($\pm$ 0.1). More accurate measurements of porosity may be made by analyzing a soil sample.

Once a constant water level and inflow is achieved, the permeability is calculated using the following equation

$$K_{fs} = \frac{Q}{A}$$

Equaton 3

Where:

Q=Volume of water added over a given time
K_{fs}=field saturated conductivity
A=area of the inner ring

Shallow Well Pump-in Test

The shallow well pump-in test is currently specified as an accepted method for analysis of in-situ permeability. The test may be run using one of two methods. The first and simplest method is to use the governing equation 4 (from Boersma) with the flow rate and well characteristics of the constant head permeameter. The second, a more cumbersome method, is described below:

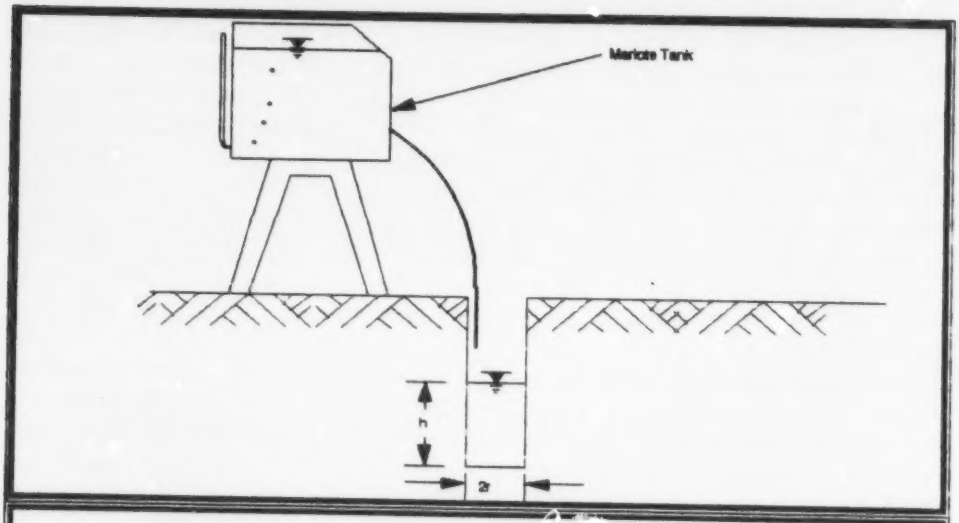

Figure 6. The Shallow Well Pump-in Test

1) A 75 mm diameter well is augured to the required depth and brushed in the same manner as the constant head permeameter procedure.

2) A coarse sand or pea stone may be placed inside the hole to stabilize the sidewalls and then water is pored into the well up to a specific height.

3) A 20 litre gas container, adapted as a Marriotte constant flow apparatus is set to a given rate and left to achieve a stable state (ie the depth of water in the hole is neither falling nor rising and is such that the outflow from the hole is equal to the inflow). The rate of inflow to the hole is measured using a measuring cylinder and stopwatch. The process can take up to 45 minutes and requires constant attention by the operator.

4) The flow rate, hole radius, and water height are then used in the equation below:

$$k = Q \frac{\ln\left[\frac{h}{r} + \sqrt{\left(\frac{h}{r}\right)^2 - 1} - 1\right]}{2\pi h^2}$$

Equaton 4

Where

k=permeability (cm/sec)
Q=inflow rate in cm^3 per sec.
r=radius of the hole (cm)
h=depth of water in the hole

Chloride Tracer Test

In 1983-1984, a monitoring project was established to assess the direction and attenuation of effluent flow from a large septic distribution trench in Port Maitland, Nova Scotia (reference 4). The study used a series of single and multilevel piezometers located at various points downslope from the distribution trench. The tracer test was chosen for the Port Maitland site as data collected from this project was readily available. Chloride is a relatively inert species and moves at a seepage velocity approximately equal to that of the pore water. By comparing the iso-concentration lines of two separate data sets, and the flow net associated with the site, it is possible to generate a good approximation of the global permeability. The value obtained can then be compared to the three previous tests. The analysis is based upon the Darcy principle, v = ki where v is the discharge water velocity, i is the hydraulic gradient and k is the permeability of the soil. The seepage velocity V_s = V/n where n is the soil porosity. Thus $K = \frac{v}{i} n$

1) The 50 mg/l iso-concentration line is plotted for June 1983 along with May 1984.

2) Two points are selected and the hydraulic gradient measured between them using a flow net of the region.

3) The migration distance is measured and then the conductivity is calculated:

Results

Phase 1

Data collected from phase 1 of this study is shown in Appendix G. It was hoped that some correlation might exist between the mean permeability values obtained from each site. Unfortunately, more than three tests per site was necessary in order to obtain a reasonable global value. The degree of variability was so severe that three trials, located within a two meter radius, using only one of the three tests, could vary by one hundred fold. Results of one specific site location using each of the three instruments would vary in a similar manner. As a result of this great variability for a given location, it was decided to focus on two sites and run several trials with each apparatus.

Phase 2

Lot 1

The Lower Sackville site was tested using the double ring infiltrometer, the constant head permeameter and the shallow well pump-in test. Due to the speed at which the Constant Head test could be performed, 20 runs were carried out over the testing period. The Double ring test was only performed 15 times on the first lot, as was the shallow-well pump-in test. Results of the first set of data collected at this site indicated that the Constant Head and

shallow-well test were somewhat similar in their mean values, whereas the double ring test gave an average value approximately ten times greater than either of the auger hole tests. As much effort had been made to minimize any effects of smearing on the auger hole type tests, it was believed that the diameter ratio of inner to outer ring of the double ring test was too low and resulted in some water from the inner ring moving into the soil matrix in a direction other than directly vertical. As a result, water from the inner ring was believed to be migrating laterally (bulbing) and giving the appearance that the higher volume of flow was attributable to a soil of higher conductivity. It was then decided to run the double ring test another fifteen times using the same outer ring and a smaller 100mm diameter inner ring. The results of this test were extremely good with mean values quite close to the other two auger hole tests.

In all three of the tests, it was noted that despite the closeness of the mean values (for the second run) there still existed a high standard deviation which often equalled the mean. This clearly underlined the need, particularly of the till soils in the Halifax County area, for several tests per site to be performed in order to converge upon a reasonably reliable permeability value. The distribution density of this site is found in Appendices D and E.

Lot 2

The apparatus used on the Port Maitland site was the same as the latter part of the Sackville study (i.e. double ring infiltrometer with the 100 mm dia inner ring, constant head permeameter) except the pump-in test values were generated using the hole dimensions and inflow rates obtained from the Constant Head tests. Of principal note, the double ring test failed to give results which were even close to those of the constant head permeameter. In this case, the mean value for the double ring test was one hundred fold higher than that of the Constant Head test. The pump-in test yielded results which were about 50% higher than the constant head permeameter. Analysis of the chloride plume was then carried out to confirm which of the two tests was more accurate in determining the global field conductivity. The results are presented in table 1.

Table 1, Calculation of Permeability Based on Chloride Plume Migration

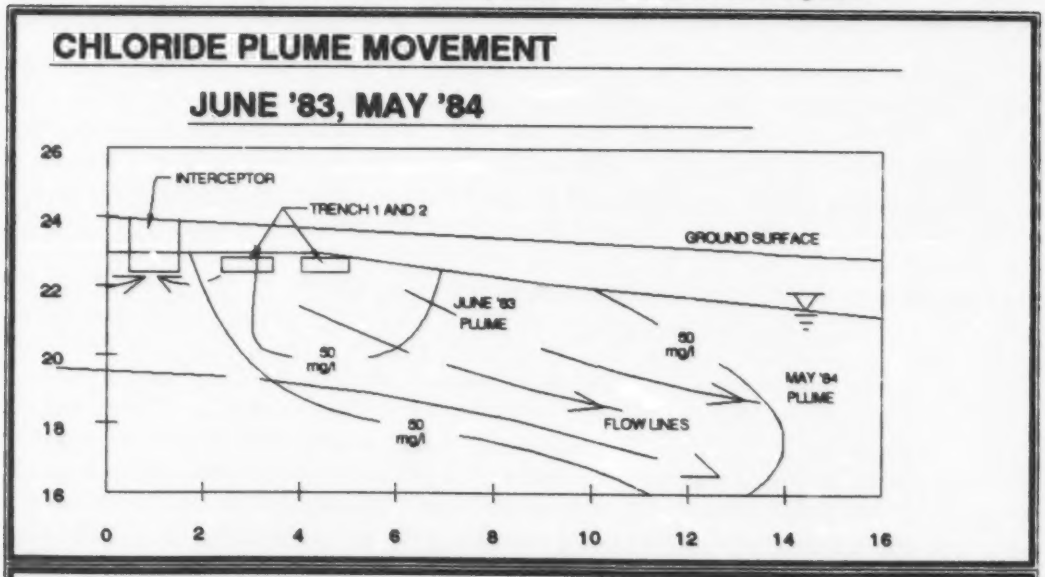

Figure 7. Chloride Plume Movement, Port Maitland

Table 1, Calculation of Permeability Based on Chloride Plume Migration

Chlorine: Distance Travelled	10 meters
Time Period	1 year
Approximate velocity of pore water:	$3.0x10^{-7}$ m/s
Hydraulic gradient	0.1
Porosity of Soil	0.45
Approximate Field Permeability	1.35x10-6 m/s

Site: Lower Sackville, Tests 1-20

	Mean ($x10^{-6}$) (m/s)	Standard Deviation ($x10^{-6}$)	# of Observations
constant head permeameter	4.85	3.04	20
Pump-In Test	4.61	2.15	20
Double Ring	60.0	60.1	20

Site: Lower Sackville, Tests 21-35

Double Ring	4.04	4.28	15

Site: Port Maitland

	Mean(m/s) ($x10^{-6}$)	Standard Deviation ($x10^{-6}$)	# of Observations
constant head permeameter	4.03	3.37	29
Pump-In Test	5.70	4.70	26
Double Ring	103.0	120.0	15
Chloride Tracer	1.35	n/a	n/a

Discussion

Based upon the data from the Sackville and Port Maitland locations, it is difficult to state with confidence whether the design of the double ring infiltrometer is adequate. The modifications made after the first twenty tests at Sackville showed that the instrument performed well, giving results consistent with that of the auger hole type test. However, a second trial with the same design at Port Maitland show a poor result with values greatly divergent from either the auger hole tests, or the chloride tracer test.

In using the equation for the shallow well pump-in test, as specified in the N.S.DoH guidelines, the in-situ permeability value is five times greater than the chloride tracer results. The Constant Head permeameter also exhibits a tendency to give permeability values higher than the global field results. The higher values obtained using the Constant Head permeameter may be a result of the "C" value used, which was taken from graphs developed from the original tests on soils which are different from Nova Scotia tills. It may therefore be necessary to modify the "C" value slightly to compensate for the local soils. Another factor which may have affected the results of the Constant Head permeameter was failing to run two different water heights in the same auger hole location. According to the Richard analysis, (reference 3) an improved estimate of the soil permeability can be made by resetting the air inlet tube so as to change the water level within the auger hole. This was attempted on three occasions in Sackville and yielded only marginal changes in the final result (typically a variation of about 10%). The comparison of the results of the preliminary

tests using two water levels and the global deviation of the soil's permeability showed that multiple readings would not significantly alter the mean value obtained for a given site.

Conclusions

The Constant Head permeameter is the most practical and portable instrument tested. More work may be necessary in calibrating the equipment to cope with the different soil types in Nova Scotia. The equation governing the pump-in test is not as accurate as the Constant Head permeameter and should be avoided, giving preference to the Constant Head analysis. The Double ring infiltrometer does not appear to give consistent results, and may provide misleading information to the designer. Furthermore, the double ring test requires too much time in set-up and monitoring, particularly in the tighter Nova Scotian till soils.

Recommendations

1) The constant head permeameter is the most reliable method for testing the global in-situ permeability of a small lot and is recommended for use in on-site assessment. However, values obtained may be slightly higher than the true global permeability of the field.

2) The shallow well pump-in test is too labour intensive and does not provide a means to adjust for differing soil conditions unlike the constant head permeameter (see figure 4). This test is recommended only if the constant head apparatus is unavailable. In providing a final analysis, the equation governing the constant head permeameter should be used.

3) To establish a reliable estimate of a given lot, a minimum of five tests is recommended (ideally in the region most likely to support the future septic bed).

4) In using the constant head permeameter, it is necessary to penetrate the ground, well below the humus and root zone.

5) The double ring infiltrometer, as documented in this study, appears to be too unreliable and cumbersome for use in the assessment of on-site permeability and is not recommended.

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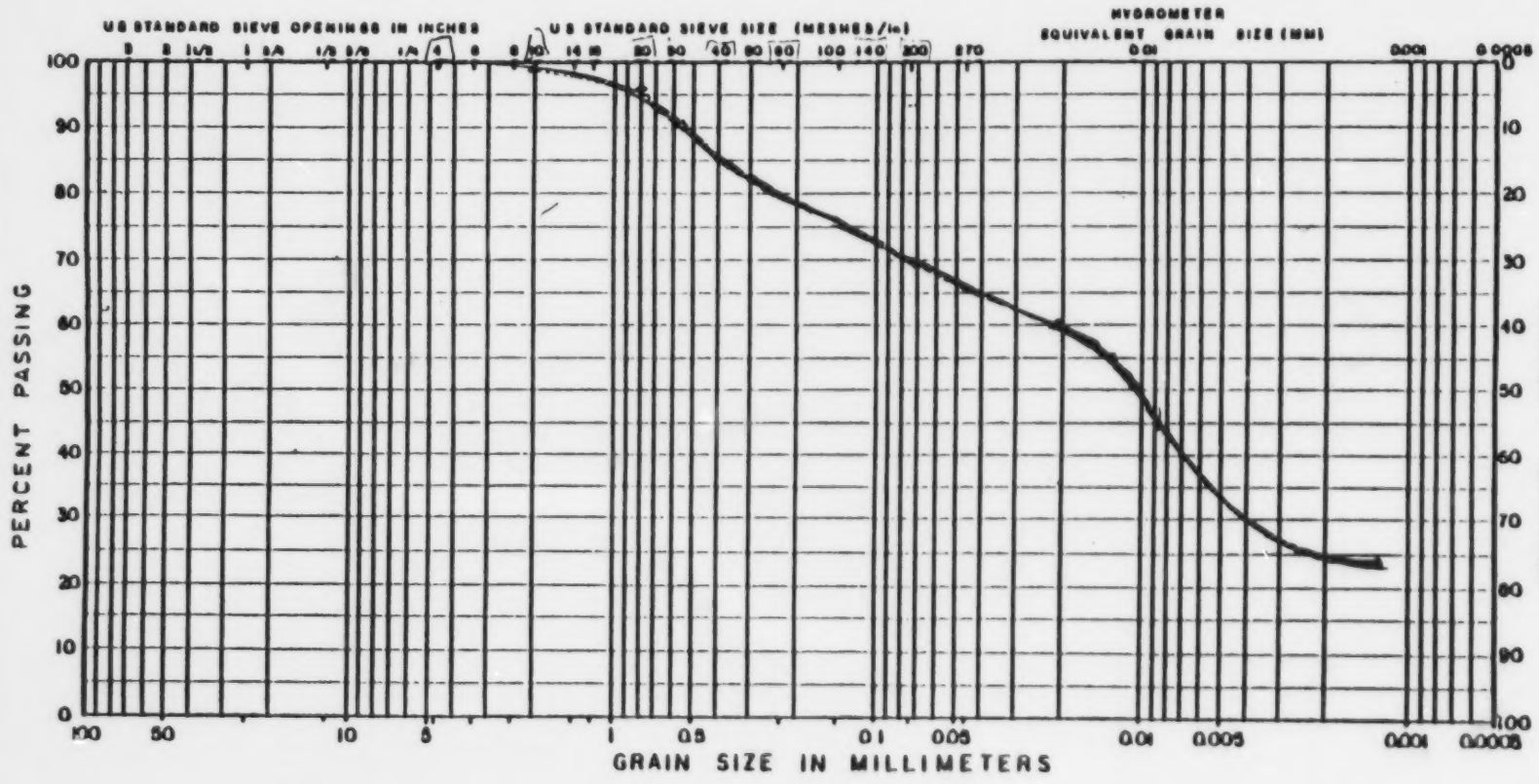
A.S.T.M. Standard Test Method for INFILTRATION RATE OF SOILS IN FIELD USING DOUBLE RING INFILTROMETERS, Designation:D 3385-75

APPENDIX A

SHEET # 3



GRAVEL		SAND			CLAY & SILT
Coarse	Fine	Coarse	Medium	Fine	

UNIFIED CLASSIFICATION

BOREHOLE No. SAMPLE No. DEPTH DESCRIPTION

GRAIN SIZE DISTRIBUTION

APPENDIX
FIGURE
PROJECT

APPENDIX B

Client **Project No.**

Project **Date**

Test Location **Test No.**

Soil Conditions

Soil Stratigraphy at Test Location .....

.....

.....

Stratigraphy determined from separate investigation pit//hole.....

Stratigraphy inferred from test location preparation

Material tested..... **Estimate porosity n =** .....

Test Apparatus and Procedure

Outer Ring I.D. .....**Inner Ring I.D.** .....**Inner Ring Area A_I** .....

Volumetric Tube I.D. .....**Area** ..... **Volume** .....cm^3/

Ring Penetration in soil .....**Outer** ..... **Inner**

Height of Water in Rings .....**h =** .....**cm(both rings same)**

Field Test Results

Trial	time	Elapsed time	tube reading	Q	d	q	K
Trial1							
Trial2							
Trial3							
Trial4							

Where: Q=Area of volumetric tube x tube reading
$d=Q/(A_I \times n)$
$q=Q/(tx60)$
$i=(h+d)d$
$k=q/(A_I \times i)$

APPENDIX C

PERMEABILITY RESULTS PORT MAITLAND

DATE	GUELPH (m/s)	DOUBLE (m/s)	PUMP IN (m/s)
AUG.4/88	3.10E-06	7.80E-06	3.46E-06
AUG.4/88	3.30E-06	1.28E-05	3.49E-06
AUG.4/88	1.20E-06	2.70E-06	4.61E-06
MAY 10/89	2.56E-06	3.19E-04	4.61E-06
MAY 10/89	2.66E-06	2.66E-04	4.61E-06
MAY 10/89	3.42E-06	3.67E-04	4.66E-06
MAY 10/89	3.42E-06	1.69E-04	4.66E-06
MAY 10/89	3.42E-06	2.29E-05	5.23E-06
MAY 10/89	3.43E-06	3.72E-06	5.77E-06
MAY 10/89	3.43E-06	7.85E-06	5.76E-06
MAY 10/89	3.79E-06	8.78E-05	5.76E-06
MAY 10/89	4.26E-06	1.48E-04	6.06E-06
MAY 10/89	4.27E-06	5.90E-05	6.49E-06
MAY 10/89	4.27E-06	1.60E-05	6.93E-06
MAY 10/89	4.46E-06	5.21E-05	7.46E-06
MAY 10/89	4.60E-06		8.07E-06
MAY 10/89	5.38E-06		1.12E-05
MAY 10/89	5.49E-06		1.84E-05
MAY 10/89	5.98E-06		2.08E-05
MAY 10/89	8.24E-06		1.92E-06
MAY 10/89	1.37E-05		9.61E-07
MAY 10/89	1.54E-05		1.67E-06
MAY 10/89	1.32E-06		2.23E-06
MAY 10/89	6.62E-07		9.51E-07
MAY 10/89	1.15E-06		1.30E-06
MAY 10/89	1.53E-06		1.12E-06
MAY 10/89	6.62E-07		
MAY 10/89	9.15E-07		
MAY 10/89	7.63E-07		

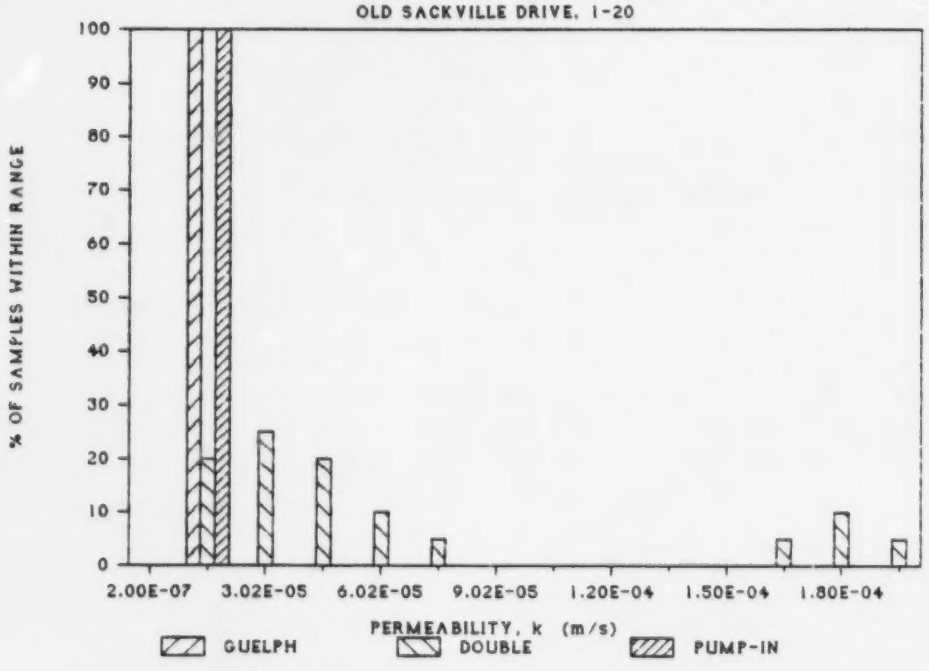
PERMEABILITY RESULTS OLD SACKVILLE DRIVE

DATE	GUELPH (m/s)	DOUBLE (m/s)	PUMP IN (m/s)
SEPT 1/88	1.30E-05	1.60E-04	4.00E-06
SEPT 1/88	5.60E-06	1.80E-04	5.40E-06
SEPT 1/88	3.55E-06	1.90E-04	5.00E-06
SEPT 1/88	5.60E-06	1.70E-04	3.00E-06
SEPT 1/88	4.60E-06	4.70E-05	6.00E-06
SEPT 1/88	8.50E-06	9.60E-06	7.80E-06
SEPT 11/88	1.20E-05	2.80E-05	5.00E-06
SEPT 11/88	3.10E-06	6.80E-05	1.20E-06
SEPT 11/88	3.00E-06	2.20E-05	7.80E-07
SEPT 11/88	3.20E-06	3.90E-05	2.00E-06
SEPT 11/88	4.00E-07	4.70E-06	3.80E-06
SEPT 11/88	2.80E-06	4.50E-05	9.00E-06
SEPT 11/88	4.40E-06	1.30E-05	3.60E-06
SEPT 11/88	4.10E-06	4.40E-05	5.00E-06
SEPT 11/88	4.30E-06	2.70E-05	7.00E-06
SEPT 11/88	3.70E-06	2.50E-05	3.00E-06
SEPT 11/88	2.50E-06	1.70E-06	8.00E-06
SEPT 11/88	6.50E-06	6.00E-05	3.30E-06
SEPT 11/88	2.70E-06	4.00E-05	5.40E-06
SEPT 11/88	3.40E-06	2.50E-05	4.00E-06
SEPT 11/88		1.30E-06	
SEPT 11/88		3.00E-06	
SEPT 11/88		2.00E-07	
SEPT 11/88		4.70E-06	
SEPT 11/88		6.70E-07	
SEPT 11/88		6.60E-06	
SEPT 11/88		3.00E-06	
SEPT 11/88		1.10E-06	
SEPT 11/88		3.70E-06	
SEPT 11/88		6.60E-06	
SEPT 11/88		8.00E-07	
SEPT 11/88		9.00E-07	
SEPT 11/88		1.80E-05	
SEPT 11/88		4.70E-06	
SEPT 11/88		5.30E-06	

APPENDIX D

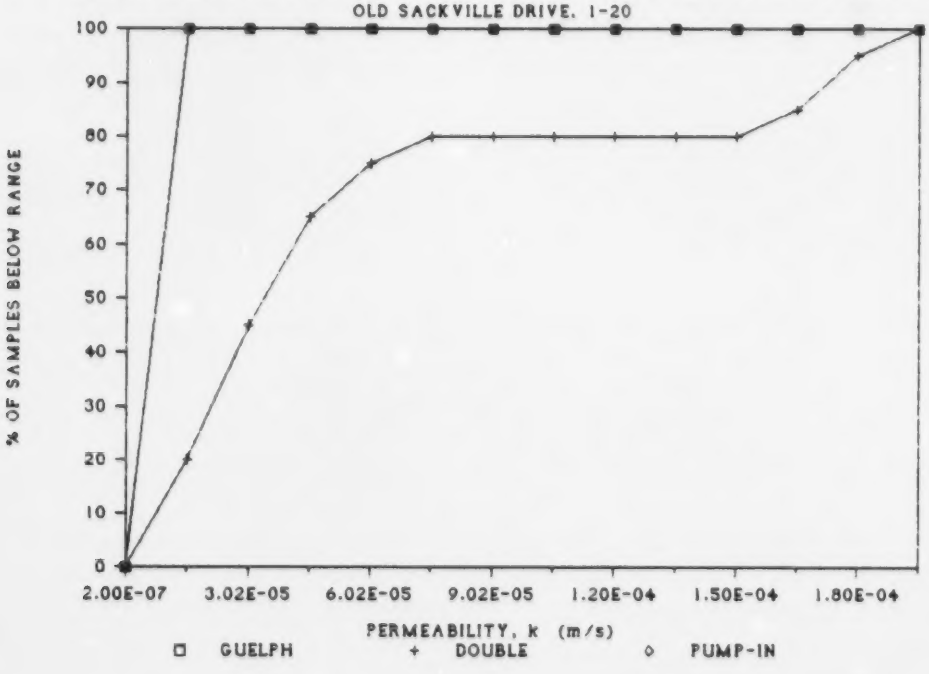
PERMEABILITY DISTRIBUTION

OLD SACKVILLE DRIVE, I-20



CUMULATIVE PERMEABILITY DISTRIBUTION

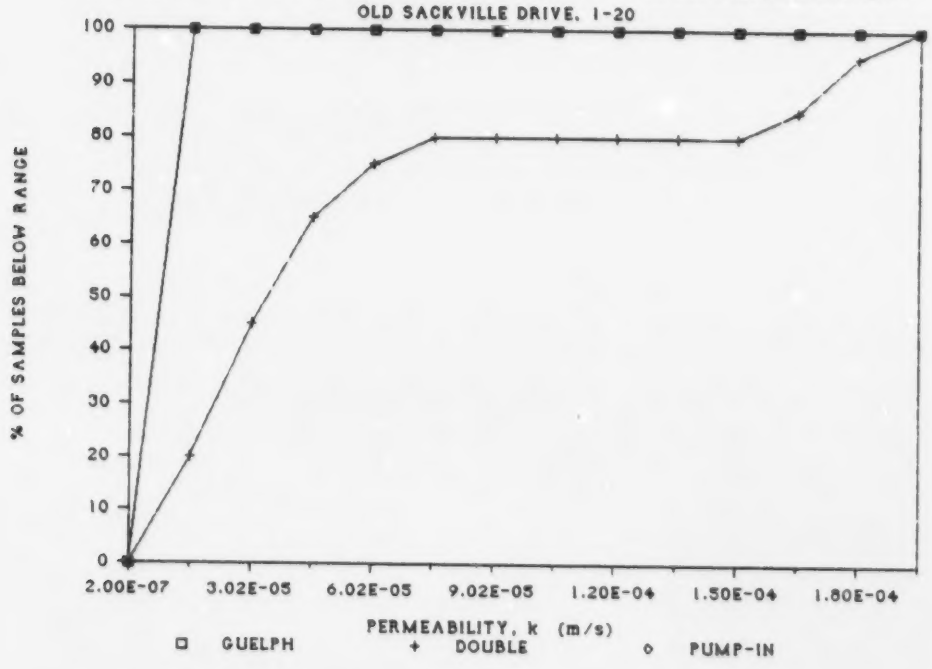
OLD SACKVILLE DRIVE, I-20



APPENDIX E

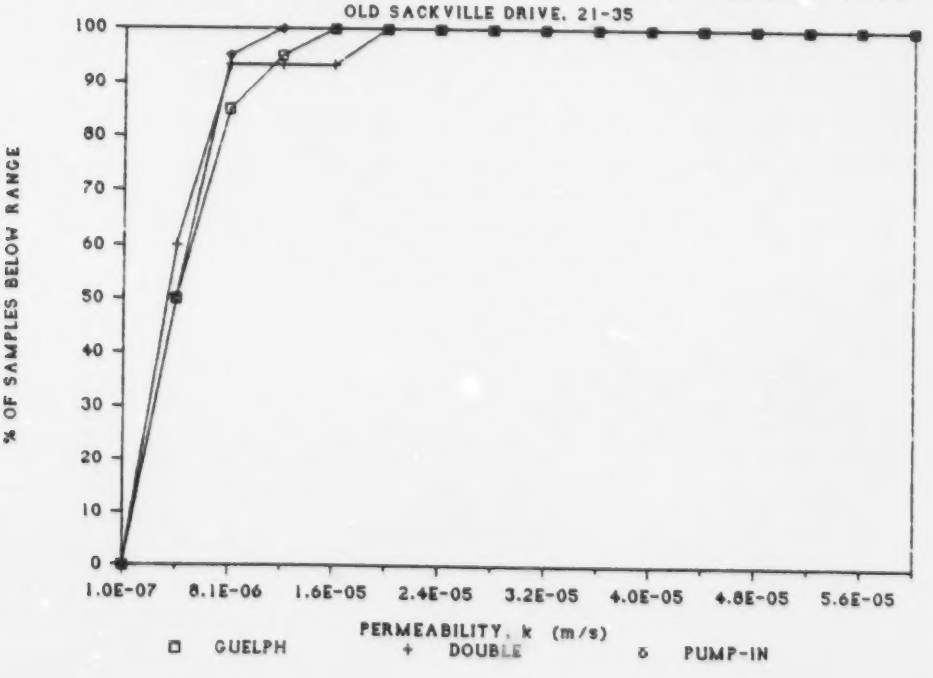
CUMULATIVE PERMEABILITY DISTRIBUTION

OLD SACKVILLE DRIVE, 1-20



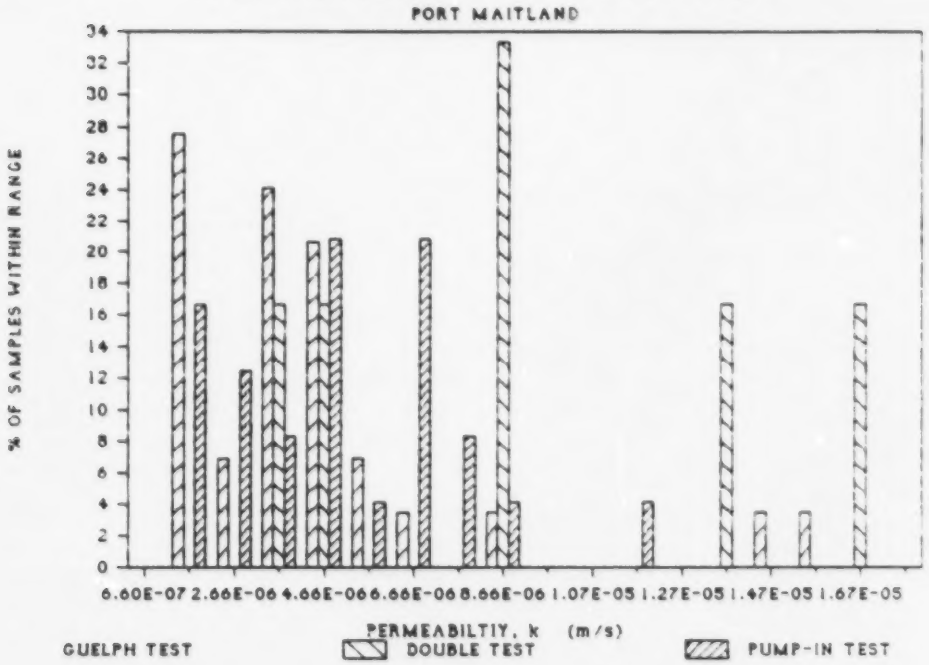
CUMULATIVE PERMEABILITY DISTRIBUTION

OLD SACKVILLE DRIVE, 21-35

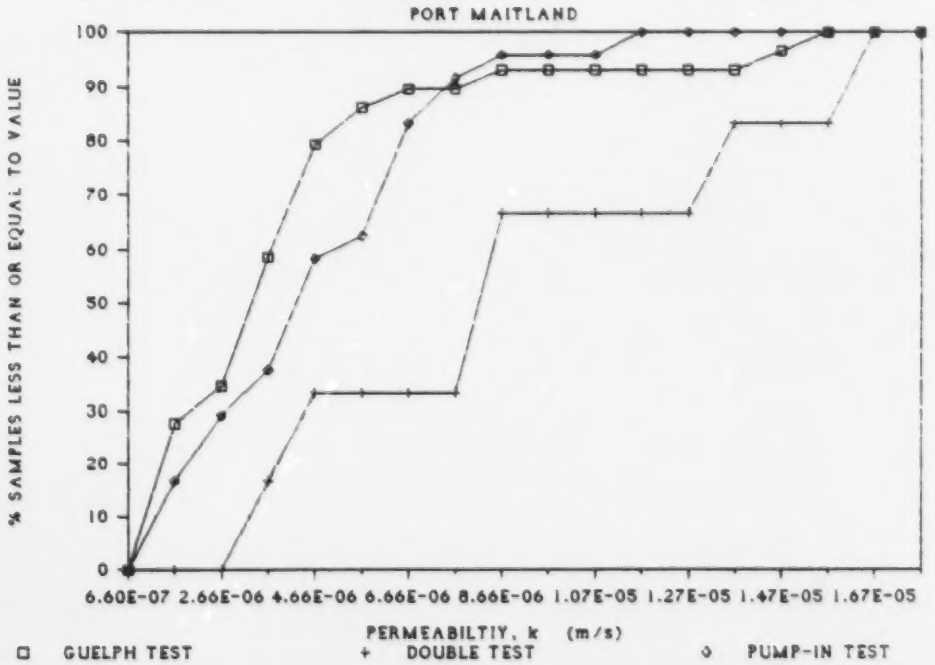


APPENDIX F

PERMEABILITY DISTRIBUTION



CUMULATIVE PERMEABILITY DISTRIBUTION



Appendix G

Data From Phase 1, Six Different Sites

Location	Region	Double Ring	Const. Head	Shallow Well	Perc Test
		m/s	m/s	m/s	m/s
1	1	2.54E-05	2.28E-05	1.87E-05	1.35E-04
1	2	5.63E-04	9.30E-06	2.55E-06	7.41E-05
1	3	1.55E-04	2.73E-05	7.77E-06	7.94E-05
2	1	2.25E-04	1.51E-05	1.17E-05	6.67E-05
2	2	1.77E-05	2.00E-05	1.29E-05	9.80E-05
2	3	3.40E-05	1.77E-07	6.83E-07	1.51E-05
3	1	3.63E-05	2.10E-06	9.00E-07	2.80E-05
3	2	3.59E-05	8.00E-06	2.10E-06	4.20E-05
3	3	1.27E-04	1.30E-06	4.13E-06	1.21E-05
4	1	4.68E-05	4.68E-06	3.53E-06	5.29E-05
4	2	7.77E-05	1.38E-06	1.54E-05	1.06E-04
4	3	5.48E-05	3.48E-06	5.54E-06	1.85E-04
5	1	6.70E-06	2.50E-06	9.50E-06	3.40E-05
5	2	5.20E-05	1.80E-06	3.20E-06	4.70E-05
5	3	1.07E-04	1.20E-05	1.20E-05	1.30E-04
6	1	7.80E-06	3.10E-06		
6	2	1.28E-05	3.30E-06		
6	3	2.70E-06	1.20E-06		

In-Situ Evaluation of Study of C2 Contour Fields

G.M. Charles, D.H. Waller

Contribution No. 90.-6

September 1990

Presented under the terms of Completion for the On-site Wastewater Treatment Program, 1987-90

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Introduction

Since the mid 1980's the Nova Scotia Department of Health and Fitness has been developing and improving the design of a series of septic wastewater disposal fields. The basic design incorporates a single line trench which evenly distributes septic wastewater into the underlying soil. Three systems have now emerged for use under the departments guidelines and are illustrated below in figure 1. Most unique of the three fields is the C2 design, which is the focus of current research by the Centre for Water Resources Studies at the Technical University of Nova Scotia.

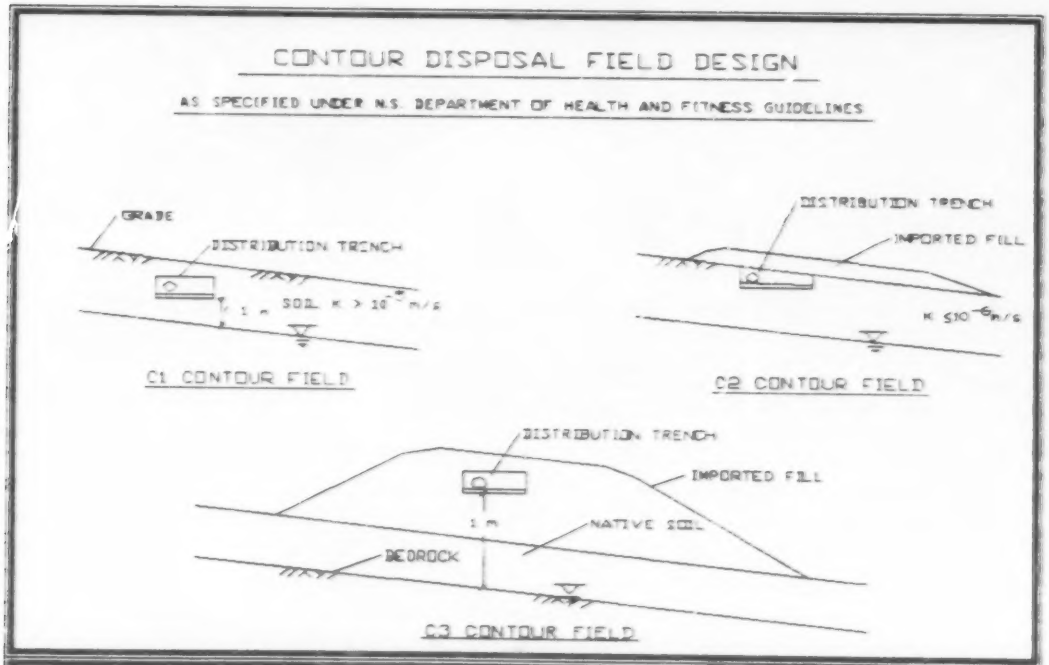


Figure 1. Various Types of Contour Fields

Scope of Research

As very little information exists regarding chemical and bacterial migration within glacial till soils, this research was carried out in two phases. The first phase of the project was designed to provide extensive groundwater quality analysis which would demonstrate the general performance of the C2 contour field. The second phase is less intensive evaluation of several other sites using similar sampling techniques as those of phase 1. This paper reports the findings of the first part of this project. All three of the sites tested were located in the Halifax County region of Nova Scotia (figure 2).

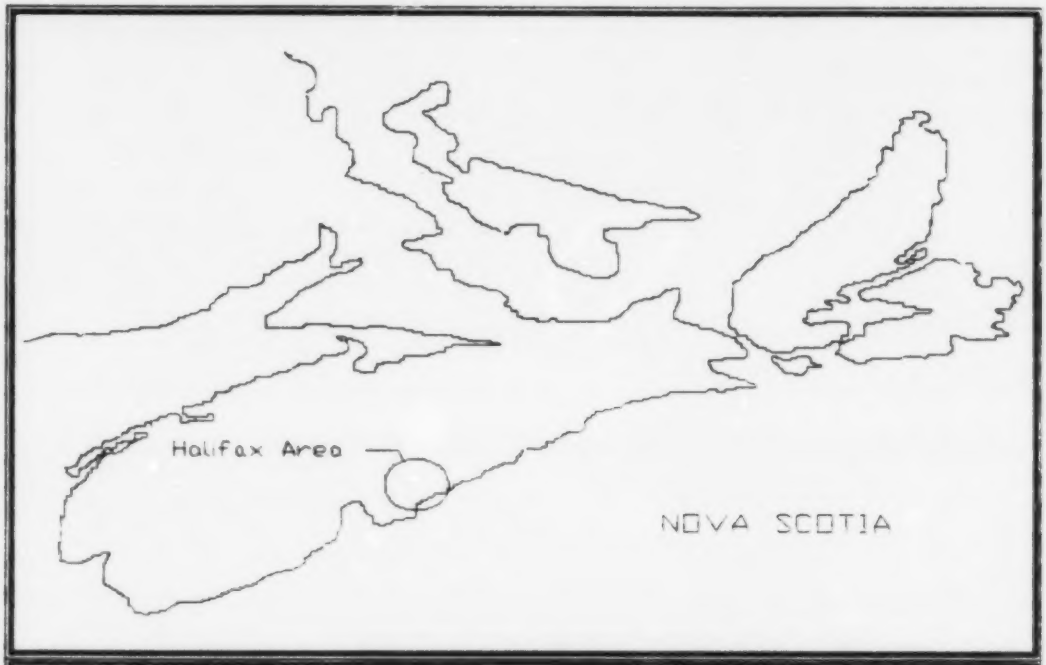


Figure 2. Region of Study

The C2 Contour Septic Field

The C2 contour septic field is a widely used method of on-site septic wastewater disposal in the province. The imported fill disposal fields used in the Province are a development of the Wisconsin mound adopted to the lateral flow concept of contour disposal fields. The C2 design is used in the lower permeability soils and is an attempt to use the natural soil profile to the best advantage. Acknowledging that a reasonable depth of glacial till will

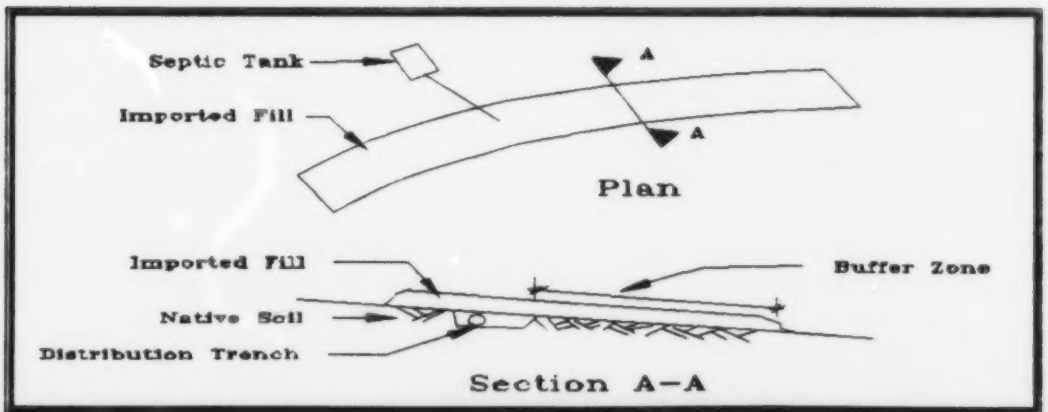


Figure 3. Typical C2 Field

effectively prevent pathogens from travelling to any local bedrock aquifer, a shallow distribution trench is set directly in the upper weathered horizon of the soil. The natural soil may initially conduct effluent under saturated conditions but capillary action in the overlying mantle of imported sand fill draws the effluent upwards. The flow then tends to become partially saturated in both the natural soil and the imported fill. This leads to effective treatment of effluent in the soil with a minimum capital cost, as there is no requirement for vertical flow through partially saturated soil, which would require a greater volume of imported fill. Figure 3 provides a general layout of the field and its various features.

The C2 field is typically used in situations where the soil permeability is less than 10^{-6} m/sec and i) water table is closer than 600 mm from the ground surface at any time throughout the year and ii) where bedrock is located further than one meter below the top of ground surface.

Methodology

Under phase one, three septic fields were selected and groundwater samples analyzed for the following:

chloride

- total organic carbon
- ammonia
- total nitrogen
- chemical oxygen demand
- biological oxygen demand
- fecal coliforms bacteria

Samples from each of the three fields were collected from wells of up to one meter in depth created by a 75 mm diameter dutch auger. Sampling depths were limited because of the high occurrence of boulders and stones in the soil which impeded the penetration of the auger. The wells were allowed to recharge with groundwater and then pumped dry using a small hand pump. Well purging was repeated a second time with the analytical sample collected on the third withdrawal. Sampling was conducted from wells located furthest from the septic trench, moving progressively closer to the trench after each collection. Chemical samples were stored in one liter plastic bottles while Fecal Coliforms samples were collected separately in sterilized 80 ml containers. The collection jar and pump tubing were cleaned with uncontaminated water between successive sampling points. The samples were returned to the laboratory where they were stored at 5°C overnight. Samples which contained larger amounts of fine sediment were decanted into a second container to provide the laboratory with a relatively clear groundwater sample. Analysis on both the Fecal Coliforms and chemical samples was begun within 24 hours.

Where appropriate, the sample results were plotted in plan elevation using a commercial contouring package for personal computers.

System Descriptions

Lot 1 Waverly, Nova Scotia

This system was operated for 1 $^{1}/_{2}$ years prior to testing. The occupants, including three children, all under four years of age, and two adults, were the principal users of the system. Periodic monitoring of the water meter at this site showed an average consumption of approximately 170 gallons per day. The lot was situated on a Stewiack till which was typical of all three sites tested. Appendix A shows a typical grain size distribution of this till. The grade of this lot was approximately five percent. The overall length of the system was 30 meters with a septic distribution trench one meter wide.

The system was chosen because of its relatively young age, and high loading rates. This field had been monitored in 1988 through 1989 for groundwater position and it was found that the water table fluctuated from 600 mm below grade in the spring to levels greater than 1200 mms.

Lots 2 and 3, Lakelands Subdivision

Both lots two and three were constructed several years ago and have been operated

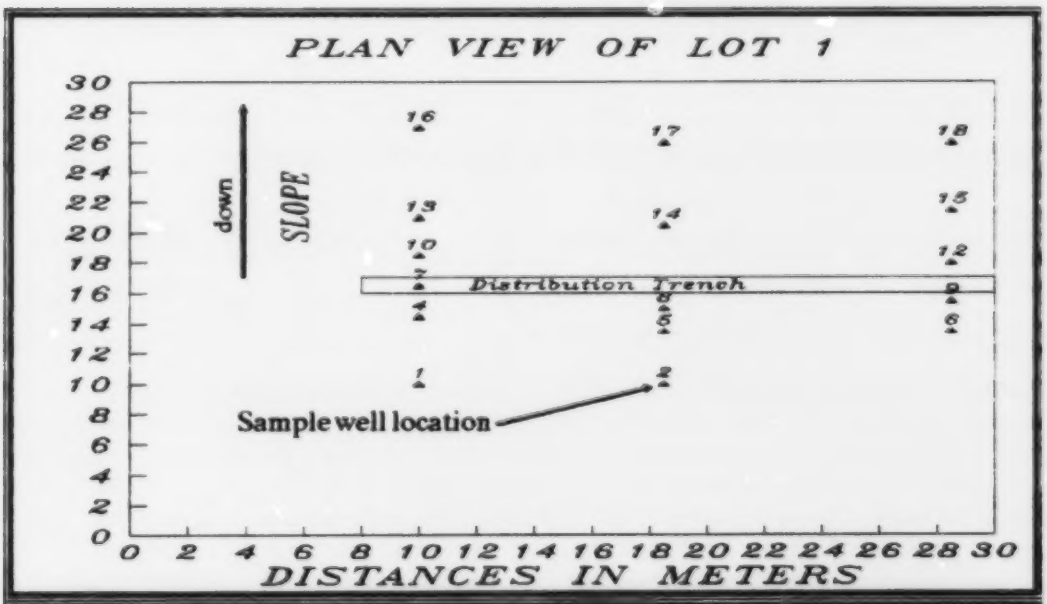

PLAN VIEW OF LOT 1
down
SLOPE
Distribution Trench
Sample well location
DISTANCES IN METERS
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30

Figure 4. Plan-Lot 1

continuously since. The description of lots two and three are listed in table 1. Both lots 2 and 3 had a groundwater table positioned quite close to the surface. Groundwater position of lot 2 had been surveyed and tested both in the fall of 1988 and again in the spring of 1989. In both instances, the groundwater table of lot 2 was 600 mm below the ground surface in the region of the imported fill, and 300 mm below the ground surface at points down-slope from the field. Lot 3 was tested only once in the spring of 1989 and it was found that the groundwater table was positioned only 450 mm below the ground surface.

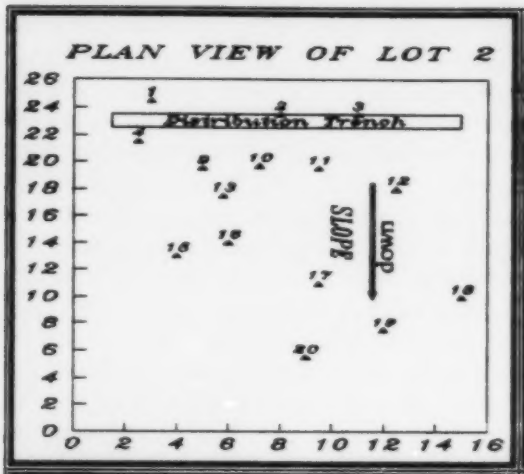


Figure 5. Plan- Lot 2

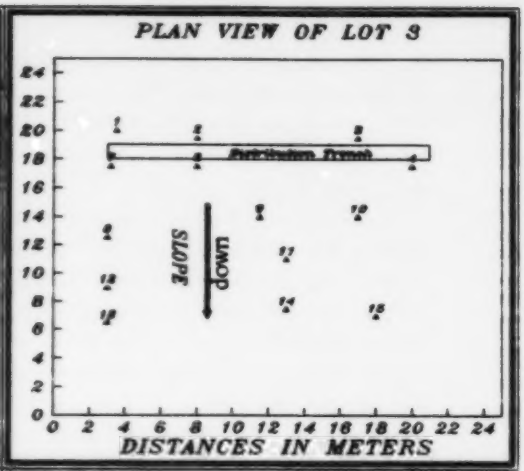


Figure 6. Plan- Lot 3

Table 1: Summary of Lots 1-3

Lot	Age of Site(yrs)	Grade (%)	Length of Trench(m)	Buffer Zone length(m)	Depth to Ground-Water	Total Occupants	Age Children	of Children (Years)
1	1.5	5	30	6	600mm	5	3	1-4
2	3.5	14	35	6	600mm	4	2	4
3	4	4	35	6	450mm	4	2	3-5

Results

Chemical Analysis

A summary of the various plots generated for each lot may be found in Appendix B. These plots were created using a computer package "Surfer". The chemical contour maps were generated using a grid formulation technique called Kriging which assumes an underlying linear variogram to be used in the interpolation of values between the various data points. It can be readily seen that although no consistent pattern emerges between the various locations for a given chemical species, certain comments can be made as to the ability of the C2 contour fields ability to treat septic wastewater. Samples of the wastewater exiting the septic tank at lot 1 were obtained using a secondary collection chamber located between the septic tank and field; analytical results are found in Appendix C. It is assumed that the septic wastewater inflow loading rates and chemical and biological constituents were similar for all three fields.

Chloride

Chloride analysis was considered in this study principally as a tool to identify the active regions and migration patterns within the septic disposal field. Chloride analysis was not

carried out on lot 1. The best transport path was generated from springtime results of lot 2. Here, extremely high chloride levels were introduced to the soil from a water softener whose backwash cycle passed directly into the septic tank. Wastewater appears to migrate in a down-slope direction, away from the septic field. Chloride distribution patterns were not as obvious on lot 3 as the chloride levels in the region were low and relatively uniform.

Nitrogen Compounds

Upon review of the total Kjeldahl nitrogen of each of the three fields it can be seen that most of the nitrogen is in the form of total organic nitrogen or ammonia. Nitrates were only present in trace amounts (under 10 mg/l) on lot 2 and were not detectable (under 0.5 mg/l) on the other two sites. The two older septic fields (lots 2 and 3) showed relatively low levels of total organic nitrogen within the disposal field area while the newer lot showed slightly higher values. Lots 1 and 3 both showed ammonia levels below 3 mg/l in the area of the field except within the vicinity of the distribution trench itself where levels were higher. Lot 3 showed slightly higher ammonia levels with an average of 10 mg/l. Comparison of ammonia levels of lot 2 from the fall of 1988 with those obtained the following spring indicate no real change in either production of ammonia or nitrates.

Total Organic Carbon

Total organic carbon was widely variable around each of the three sites. It was found that a plot of the T.O.C. levels of sites 1 and 3 showed no attenuation patterns with distance from the septic distribution trench. However, results for lot two, obtained in the spring, did show a noticeable decline in total organic carbon with increasing distance from the field.

Biological Oxygen Demand

BOD was evaluated on only two of the three lots and was found to be in the range of 0 to 10 mg/l. The first lot was not sampled due to low sample volumes.

pH

The groundwater in this type of soil tends to be slightly acidic. The pH on all three sites increases slightly but returns to lower values with increasing distance from the distribution trench.

Bacterial Analysis

Of the three sites sampled, only lot 1 showed 100 % removal of fecal coliforms. It is seen from the data set of Appendix C that the fecal coliforms levels drop from levels of 0.65-4.5 million mpn/100 mls to zero over a very short distance. Lot 2 and 3 both showed signs of fecal coliforms at varying locations from the distribution trench. These results are not unexpected for lot 2 as a close examination of the water table position revealed that the distribution trench was slightly above the water level of the interceptor and some reverse flow did occur. This caused a malfunction of the field via a backflow of effluent into the interceptor.

Ground Water Location

The groundwater position at the time of testing is shown in figure 4. Only lot 1 had a groundwater table which appeared to be below the bottom elevation of the distribution trench. Lot 2 appeared to have flow in two directions which may have been a result of groundwater "mounding" below the septic trench which induced gradients favouring bilateral flow. It is possible that the added loading by the water softener further increased this phenomena. As a result, the interceptor trench of lot 2 received untreated septic wastewater which appeared at locations downslope from the field. Lot 3 had a similar backflow problem but was not aggravated by a water softener.

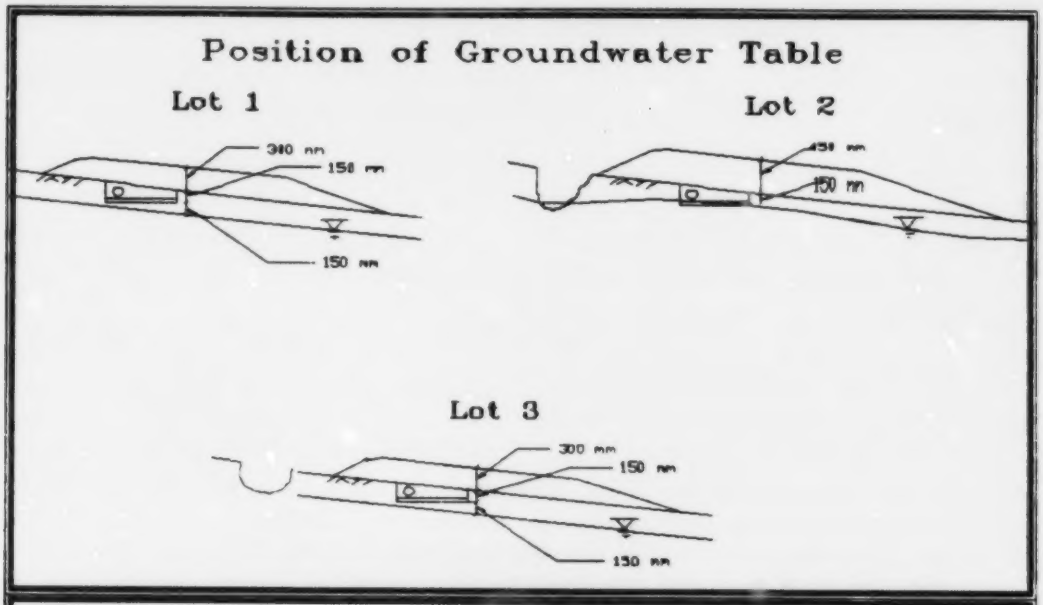


Figure 7. Groundwater Position- 3 Locations of Study

Discussion

Field Performance

With the exception of the back-flow condition arising on lots 2 and 3, the C2 design appears to work well. A comparison of the average inflow and outflow levels of the various chemical components show there is good all-around treatment. This type of septic field design produces a minimum amount of nitrate and low levels of ammonia. It is uncertain if the C2 design is deficient in its ability to remove final amounts of organic nitrogen discharged from the septic tank. T.K.N. appears to be mostly in the form of organic nitrogen which may be from either septic wastewater or natural decomposition products found in the host soil. With the exception of lot 2, total organic carbon does not necessarily decline in concentration with distance from the distribution trench. However, T.O.C. levels in the groundwater of all three sites were, in general, about one half the inflow level, indicating some T.O.C. reduction. Biological Oxygen Demand was quite low in both lots 2 and 3, indicating a low concentration of biodegradable species. It could be inferred that low BOD levels may also be indicative of a good removal of various bacterial species.

	units	Inflow mean	Site 1	Site 2 Fall 1988	Site 2 Spring 1989	Site 3
Nitrate	mg/l	<0.05	<0.05	<0.05	<0.05	<0.05
Ammonia	mg/l	44.7	5.6	7.4	10.1	0.8
T.K.N.	mg/l	61.8	42.9	89.9	22	4.9
T.O.C.	mg/l	62.6	41.4	34	36.2	26.1
B.O.D.	mg/l	119	n/a	n/a	10	5.4
Organic Nitrogen	mg/l	17.1	37.3	71.9	11.9	4.1
Chloride	mg/l	80	n/a	n/a	1107	18.6
pH		7.3	5.9	6.9	6.5	6.0
FecalCol.	[illegible]			329		

Table 2: Averages of Results

Lot 2 showed higher than normal levels of chloride during the spring which is attributed to a the water softener backwash cycle. This extra water loading may contribute to flow rates which exceed the capacity of the C2 design and its ability to adequately treat septic wastewater. Lot 3 operates under similar conditions as lot 2 in that a high groundwater table persists at differing times throughout the year. Unlike lot 2, lot 3 has no water softening outflow to the septic tank and only a limited occurrence of fecal coliforms. As both the groundwater table and soil of lots 2 and 3 are similar, it can be assumed that a C2 septic field can remove fecal coliforms providing the system does not become overloaded by water softeners. Lot 1, which operated under similar soil conditions as the other two, showed excellent bacterial removal which could be attributed to a lower water table position and no backwash water from a water softener.

Conclusions

C2 contour septic fields provide a reasonable alternative in the treatment of septic wastewater in glacial till soils. The C2 design will continue to remove fecal coliforms bacteria even if there is not a large separation distance between the groundwater table and the septic wastewater distribution trench. Nitrogen appears to be predominantly in the form of organic nitrogen or ammonia; nitrates are not formed in any large amounts in the immediate area of the C2 field. It is not certain whether nitrification will occur at points more distant from this type of field. The pH of the field is increased slightly in the vicinity of the septic trench but returns to background levels after about 10 meters from the distribution field. Further, the C2 field should not receive the water expelled during the backwash cycle of a water softener as it may induce groundwater mounding and bilateral flow away from the septic trench, carrying untreated waste to the interceptor. If the water table comes into contact with the septic trench, there will be an increased likelihood that fecal coliforms will be transported in the interceptor trench. To minimize potential backflow of wastewater into the interceptor trench, special attention during the design and construction phase must be given to ensure that the floor of the interceptor trench is well above the grade of the wastewater trench.

Implications

Existing C2 systems were monitored to determine their performance and yielded the following results:

- C2 septic fields have proven effective in removing BOD, COD, Organic Carbon and fecal coliforms from septic effluent.
- In fine soils exposed to high annual rainfall (~1200 mm/yr), separation distances between the distribution trench and the groundwater table can be in the order of 200 mm to provide adequate treatment of septic tank effluent.
- Nitrification is not occurring in the C2 systems or in the soil immediately adjacent to the system.
- Investigation of a C2 system serving a household which discharged the backwash from its water softener into the septic field indicated hydraulic overloading leading to partial malfunction as well as extremely high Chloride levels (in excess of 4000 mg/l)in the surrounding groundwater table.
- To prevent the distribution trench from back-flowing to the interceptor drain, a minimum separation distance of at least three meters is required.

References

1.N.S. Department of Health and Fitness Onsite Sewage Disposal Systems Technical Guidelines, Feb. 1988

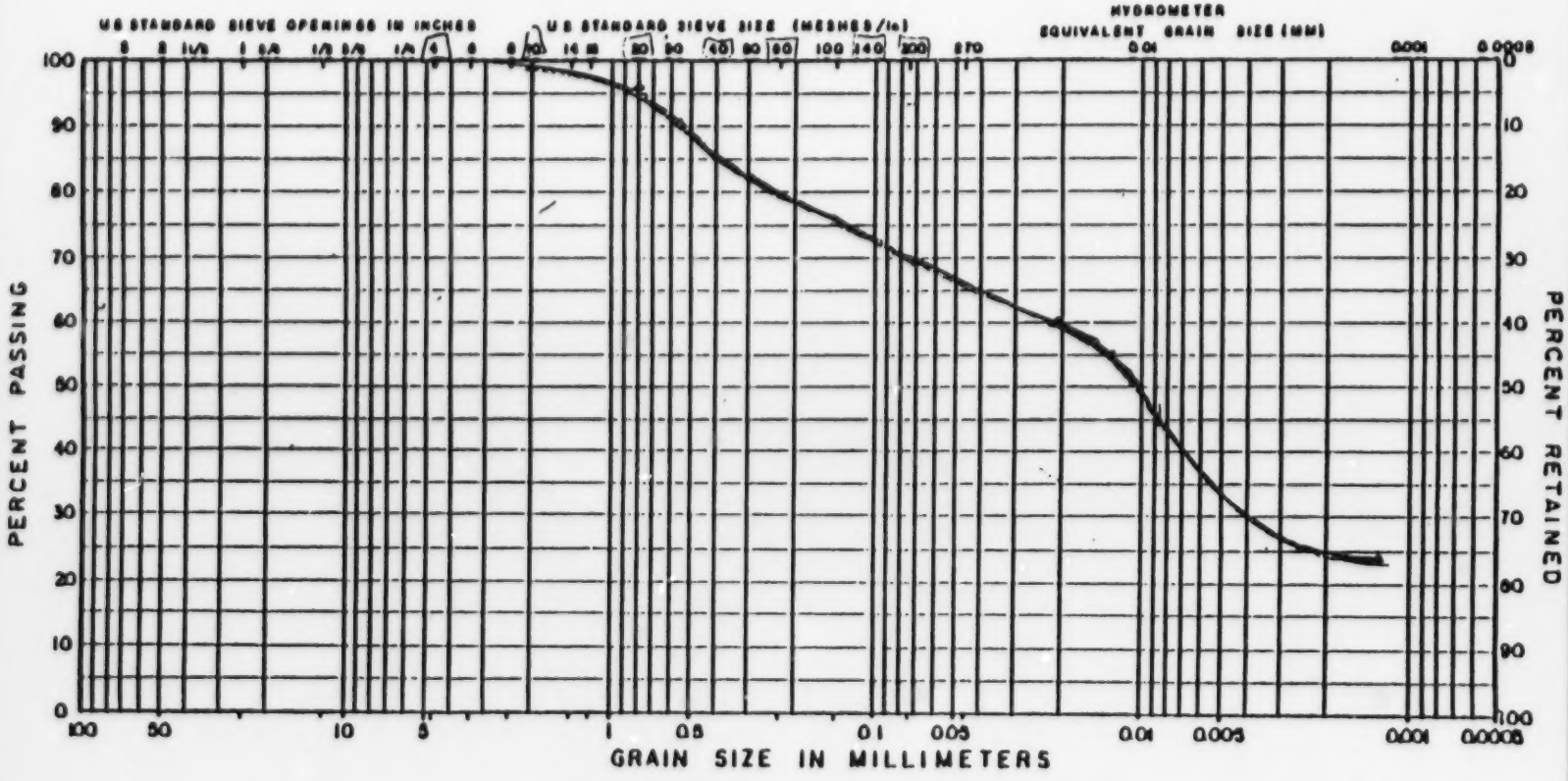
2.U.S. Environmental Protection Agency "Design Manual- On-Site Wastewater Treatment and Disposal Systems", October 1980, EPA 625/1-80-012.

3.Canter and Knox, Septic Tank System Effects on Ground Water Quality, Lewis Publishers, 1985

Design Manual- On Site Wastewater Treatment and Disposal Systems, 1980 USEPA

Appendix A

SHEET # 3



GRAVEL		SAND			CLAY & SILT
Coarse	Fine	Coarse	Medium	Fine	

UNIFIED CLASSIFICATION

BOREHOLE No	SAMPLE No	DEPTH	DESCRIPTION

GRAIN SIZE DISTRIBUTION

APPENDIX
FIGURE
PROJECT

Appendix B

Chemical and Bacterial Inflow Data From Lot 1

DATE	NO3	KJELDAHL	NH3	ORG.NIT.	TOC	COD	BOD	FECAL COL	PH
01-Jun-88	0.1	9.8	1.0	8.8	48.5	359.0		600000	
06-Jun-88	0.1	38.0	28.0	10.0	43.0	291.0		20000	
14-Jun-88	0.1	44.0	36.0	8.0	69.0	400.0		130000	
27-Jun-88	0.1	57.0	37.0	20.0	50.0	322.0		2400000	
06-Jul-88	0.1	61.0	48.0	13.0	50.0	302.0		1400000	
08-Jul-88								2400000	
15-Jul-88	0.1	130.0	49.0	81.0	82.0			220000	
19-Jul-88		205.0	37.0	168.0	62.0	330.0		2500000	
27-Jul-88	0.1	66.0	55.0	11.0	95.0	442.0		2800000	
03-Aug-88	0.1	62.0	51.0	11.0	78.0	420.0		240000	
07-Aug-88	0.1	115.0	47.0	68.0	65.0			2500000	
15-Aug-88	0.1	46.0	36.0	10.0	45.0	336.0		60000	
22-Aug-88	0.1	50.0	43.0	7.0	48.0	213.0	59.0	3000000	
28-Aug-88	0.1	60.0	48.0	12.0	44.0	252.0	98.0	200000	
09-Sep-88	0.0	65.0	52.0	13.0	38.5	510.0		60000	
12-Sep-88	0.1	65.0	61.0	4.0	100.0	457.6	202.0	200000	
19-Sep-88	0.1	55.0	47.0	8.0	80.0	332.8	126.0	450000	
26-Sep-88	0.1	45.0	44.0	1.0	36.0	182.0	48.0	110000	
03-Oct-88	0.1	53.0	46.0	7.0	56.0	390.0	181.0	90000	
13-Oct-88	0.1	65.0	55.0	10.0	93.0	572.0	174.0	300000	7.3
19-Oct-88	0.1	60.0	52.0	8.0	105.0	416.0	157.0	560000	7.3
24-Oct-88	0.1	52.0	44.0	8.0	52.0	296.8	94.0	200000	7.2
31-Oct-88	0.1	44.0	40.0	4.0	106.0	319.0	166.0	50000	6.9
07-Nov-88	0.6	54.0	43.0	11.0	71.0	285.0	122.0	170000	
16-Nov-88	0.5	42.0	37.0	5.0	44.0	201.6	70.0	700000	7.6
21-Nov-88	0.5	51.0	40.0	11.0	52.0	330.4	112.0	1200000	7.4
25-Nov-88	0.5	51.0	40.0	11.0	52.0	330.4	112.0	1200000	7.4
28-Nov-88	0.1	47.0	42.0	5.0	62.5	301.6	273.0	240000	7.3
04-Dec-88	0.5	52.0	47.0	5.0	25.0	166.4	45.8	60000	7.3
05-Dec-88	0.5	52.0	47.0	5.0	25.0	166.4	45.8	60000	7.3
12-Dec-88	0.1	47.0	39.0	8.0	39.0	184.8	58.7	170000	7.2
27-Feb-89	0.5	39.0	31.0	8.0	67.0			60000	7.4
07-Mar-89								60000	
14-Mar-89	0.5	43.0	35.0	8.0	44.0			700000	7.2
21-Mar-89								40000	
28-Mar-89	0.5	38.0	37.0	1.0	45.0			40000	7.3
04-Apr-89								40000	
11-Apr-89	0.5	65.0	56.0	9.0	50.0			10000	7.3
21-Apr-89								40000	
26-Apr-89	0.1	51.0	47.0	4.0	137.0			2000	7.1
05-May-89								20000	
09-May-89	0.5	47.0	38.0	9.0	42.0			50000	7.8
AVERAGE	0.2	61.8	44.7	17.1	62.6	323.3	119.1	511167	7.3
STDEV	0.2	32.3	7.0	32.2	26.1	109.3	61.3	829905	0.2

Lot 1, Spring 1989

LOTNUM	WELLNUM	CODVALUE (mg/l)	TOCVALUE (mg/l)	TKNVALUE (mg/l)	AMMONIA (mg/l)	ORG.NIT. (mg/l)	NITRATE (mg/l)	FECALCOL mpn/100 ml	PHVALUE
1	1	717	2.8	14.5	0.5	14.00	0.5	0	4.8
1	2	4144	82.0	60.0	2.7	57.30	0.5	0	5.5
1	4	1848	0.9	35.5	0.5	35.00	0.5	0	5.1
1	5	7560	87.0	175.0	1.3	173.70	0.5	0	6.9
1	6	728	24.0	15.0	0.7	14.35	0.5	0	7.1
1	7	952	47.0	63.0	27.0	36.00	0.5	1000000	7.1
1	8	297	73.0	37.5	31.0	6.50	0.5	4500000	7.2
1	9	269	58.0	25.0	18.0	7.00	0.5	650000	7.7
1	10	896	5.1	21.0	0.5	20.50	0.5	0	5.2
1	12	3612	109.0	73.0	2.5	70.50	0.5	0	6.1
1	13	538	7.9	10.5	0.5	10.00	0.5	0	4.7
1	14	431	27.0	12.0	0.1	11.95	0.5	0	5.1
1	15	1680	27.0	31.0	3.5	27.50	0.5	0	6.1
1	16	370	7.6	10.0	0.5	9.50	0.5	0	5.1
1	17	4536	90.0	93.0	0.5	92.50	0.5	0	4.7
1	18	392	14.0	10.5	0.5	10.00	0.5	0	5.2
Mean		1811	41.4	42.9	5.6	37.3	0.5	384375	5.9
Standard Deviation		2022	35.5	42.1	9.8	42.9	0.0	1098539	1.0

Lot 2, Fall 1988

Lot #	LOCATION	NITRATE (mg/l)	AMMONIA (mg/l)	pH	T.O.C. (mg/l)	C.O.D. (mg/l)	T.K.N. (mg/l)	ORG. NIT (mg/l)	FECAL mpn/100 mls
2	2	0.5	8.5	7.0	86.0	4394.0	84.0	75.5	0
2	3	0.1	10.0	7.8	38.5	109.2	11.3	1.3	0
2	4	1.5	6.2	7.3	35.5	135.2	18.0	11.8	0
2	5	10.0	1.3	6.8	34.5	10752.0	215.0	213.7	0
2	6	1.0	1.6	7.4	22.1				0
2	7	0.1	14.0	6.6	41.5				0
2	8	3.0	4.9	7.2	16.1	832.0	27.0	22.1	0
2	9	2.4	2.6	6.8	12.0	313.6	13.5	10.9	0
2	10	3.4	13.0	7.8	31.8	436.0	20.0	7.0	0
2	11	0.1	7.5	6.8	100.0	26600.0	655.0	647.5	0
2	12	0.1	11.0	6.5	40.0	4420.0	100.0	89.0	60000
2	13	1.1	1.0	4.9	5.0	2352.0	56.0	55.0	0
2	14	5.7	4.7	7.4	18.6	134.4	6.0	1.3	0
2	15	4.1	4.8	7.4	17.4	145.6	10.0	5.2	1000
2	131	0.1	10.0	6.6	17.8	1400.0	50.0	40.0	1000
2	132	0.1	12.0	6.4	43.0	6160.0	58.0	46.0	
2	133	0.1	13.0	6.7	17.6	1568.0	25.0	12.0	
MEAN		1.9	7.4	6.9	34.0	3983.5	89.9	82.6	4133
STD. DEVIATION		2.6	4.3	0.7	24.4	6697.9	159.7	160.0	14935

Lot 2, Spring 1989

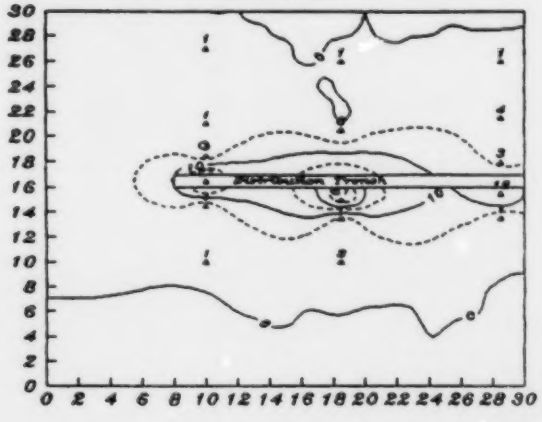
LOTNUM	WELLNUM	BODVALUE (mg/l)	CHLORIDE (mg/l)	TOCVALUE (mg/l)	TKNVALUE (mg/l)	AMMONIA (mg/l)	ORG. NIT. (mg/l)	NITRATE (mg/l)	FECALCOL (mpn/100 ml)	PHVALUE
2	1		1188.0	66.0	17.0	12.8	4.2	<0.5	300	7.1
2	2		1447.0	24.0	11.0	8.1	2.9	<0.5	0	5.8
2	3		3526.0	102.0	43.0	42.0	1.0	<0.5	1000	7.1
2	7		951.0	60.0	19.0	14.5	4.5	<0.5	180	7.8
2	9		942.0	39.0	13.0	11.5	1.5	<0.5	120	7.1
2	10		1243.0	40.0	17.0	13.6	3.4	1.4	0	8.0
2	11		2032.0	40.0	24.0	18.6	5.4	0.3	1900	7.8
2	12		933.0	38.0	13.0	10.1	2.9	<0.5	100	7.4
2	13		767.0	38.0	14.0	10.2	3.8	<0.5	0	6.9
2	15		542.0	83.0	128.0	3.5	124.5	<0.5	0	6.0
2	16	6.3	662.0	23.0	5.2	2.0	3.2	<0.5	500	6.6
2	17	33.0	1132.0	16.0	33.0	5.0	28.0	<0.5	0	6.8
2	18	11.0	1741.0	20.0	18.0	15.6	2.4	<0.5	1000	6.9
2	19	17.7	1259.0	17.0	17.7	3.8	13.9	6.2	500	7.0
2	20	2.0	454.0	9.8	1.8	0.4	1.4	<0.5	0	6.0
Mean		10.0	1107.0	36.2	22.0	10.1	11.9	0.5	329	6.5
Standard Deviation		11.1	799.5	27.3	28.6	9.8	28.9	1.5	509	1.8

Lot 3, Spring 1989

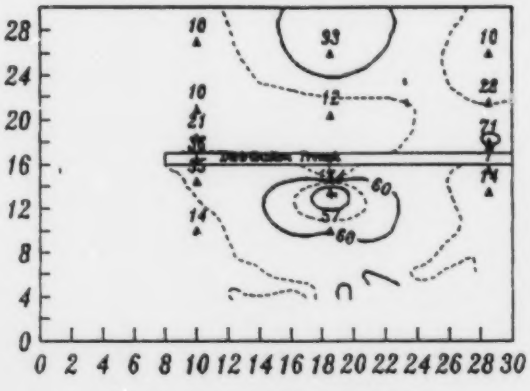
LOTNUM	WELLNUM	BODVALUE (mg/l)	CHLORIDE (mg/l)	TOCVALUE (mg/l)	TKNVALUE (mg/l)	AMMONIA (mg/l)	ORG. NIT. (mg/l)	NITRATE (mg/l)	FECALCOL mpn/100 ml	PHVALUE
3	1	2.0	20.0	13.0	1.4	0.2	1.3	<0.5	0	4.7
3	2	3.7	25.0	35.0	5.3	0.1	5.2	<0.5	0	5.5
3	3		5.2	4.2	0.8	0.2	0.6	<0.5	0	5.4
3	4	2.0	10.0	13.0	2.7	0.4	2.4	<0.5	0	5.9
3	6	2.0	21.0	13.0	2.5	0.5	2.0	<0.5	0	5.7
3	7		12.0	6.4	6.4	0.3	6.1	<0.5	0	4.2
3	8		30.0	152.0	9.0	1.0	8.0	<0.5	0	6.6
3	9	6.2	26.0	31.0	6.2	1.9	4.3	<0.5	0	6.8
3	10		20.0	15.0	6.0	3.6	2.4	<0.5	1000	7.2
3	11	16.9	22.0	25.0	4.8	0.9	3.9	<0.5	500	6.8
3	12		8.2	5.0	0.8	0.1	0.6	<0.5	0	5.2
3	13	2.3	22.0	5.0	0.5	0.1	0.4	<0.5	0	5.3
3	14	5.4	25.0	21.0	14.0	0.1	13.9	<0.5	0	6.2
3	15	8.4	14.0	27.0	8.0	1.7	6.3	<0.5	0	5.4
3	20								0	
3	21								10	
3	22								10	
3	23								0	
Mean		5.4	18.6	26.1	4.9	0.8	4.1	n/a	84	6
Standard Deviation		4.6	7.2	35.2	3.7	1.0	3.6	n/a	250	1

Appendix C

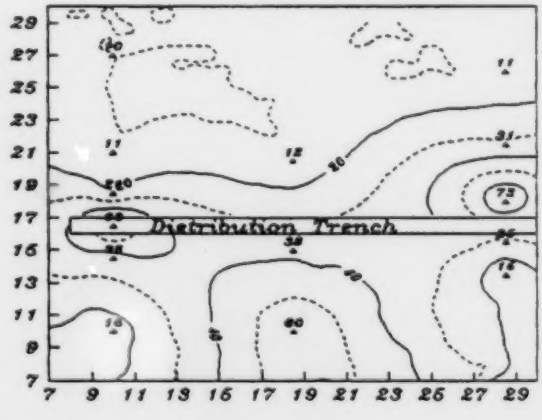
LOT 1. AMMONIA LEVELS



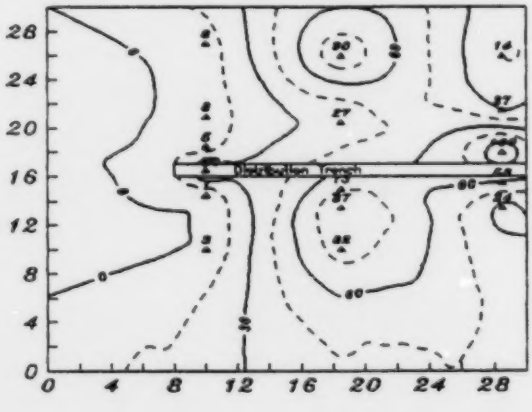
LOT 1, ORGANIC NITROGEN LEVELS



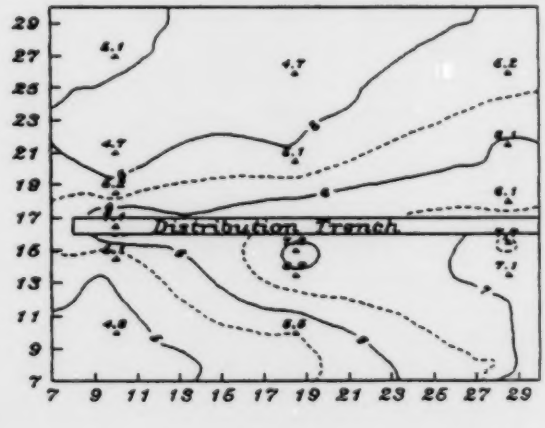
LOT 1. TKN LEVELS



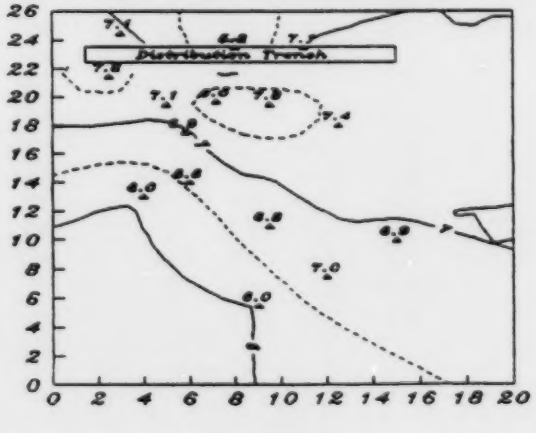
LOT 1. TOC LEVELS



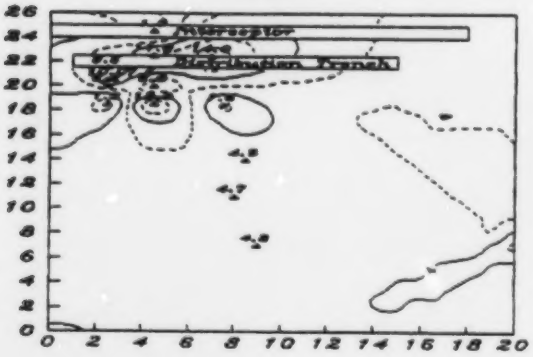
LOT 1. PH LEVELS



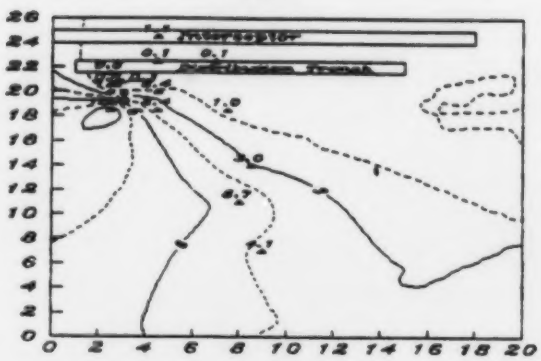
LOT 2. PH LEVELS



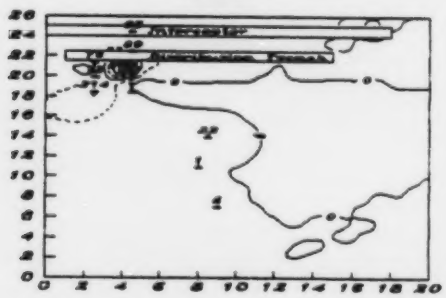
LOT 2. AMMONIA LEVELS
FALL



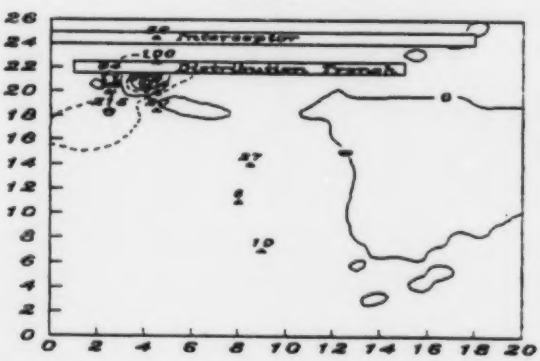
LOT 2. NITRATE LEVELS
FALL



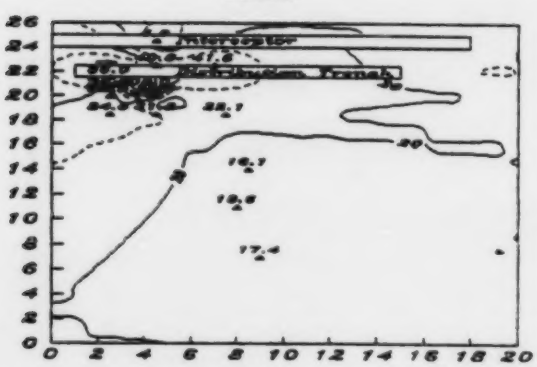
LOT 2. ORGANIC NITROGEN LEVELS
FALL



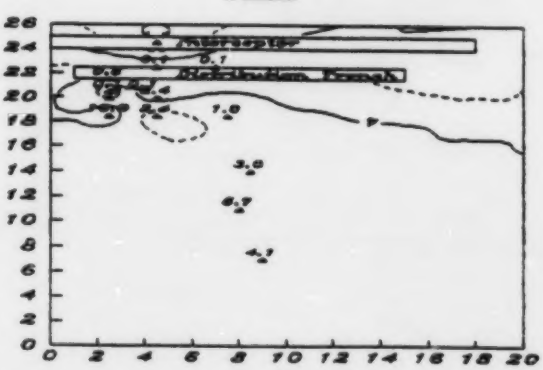
LOT 2. TKN LEVELS
FALL



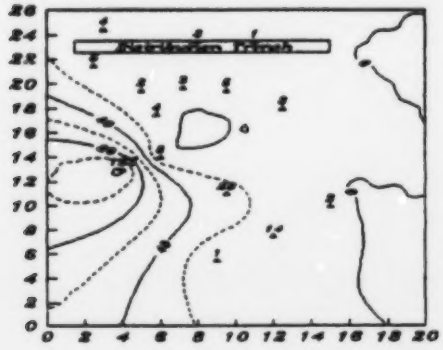
LOT 2. TOC LEVELS
FALL



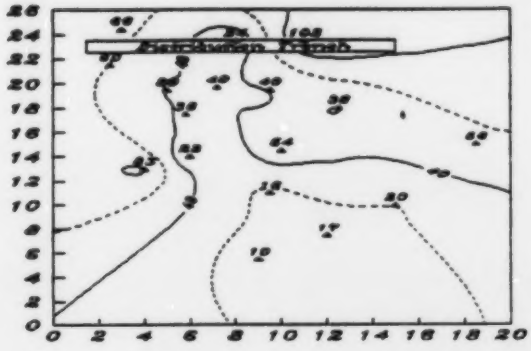
LOT 2. PH LEVELS
FALL



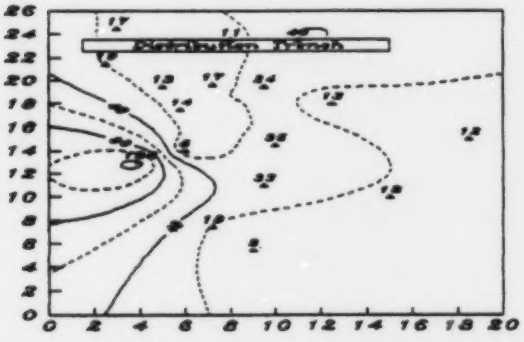
LOT 2, ORGANIC NITROGEN LEVELS
Spring



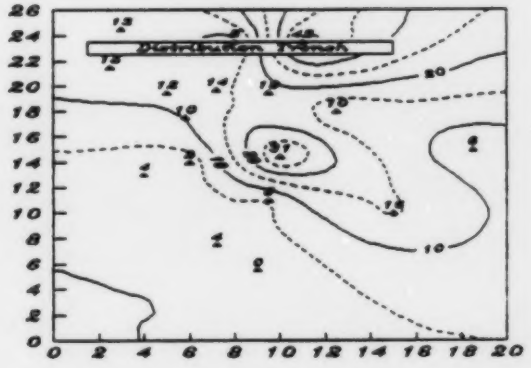
LOT 2, TOC LEVELS
Spring



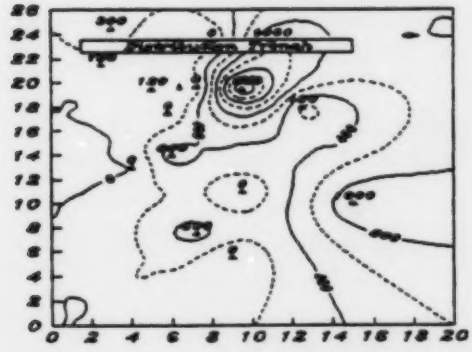
LOT 2, TKN LEVELS
Spring



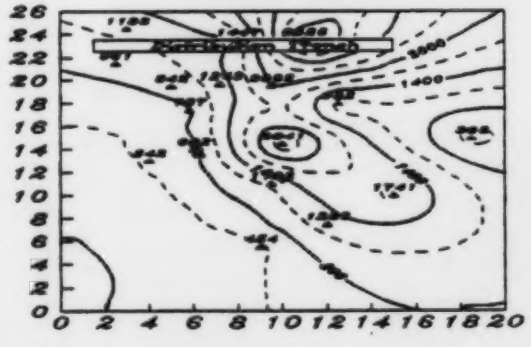
LOT 2, AMMONIA LEVELS
Spring



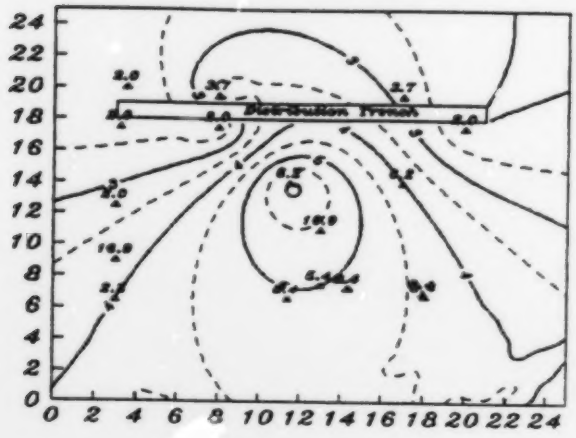
LOT 2, FECAL COLIFORM LEVELS
Spring



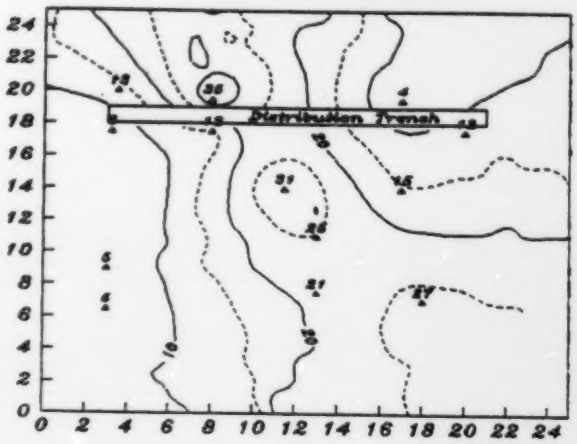
LOT 2, CHLORIDE LEVELS



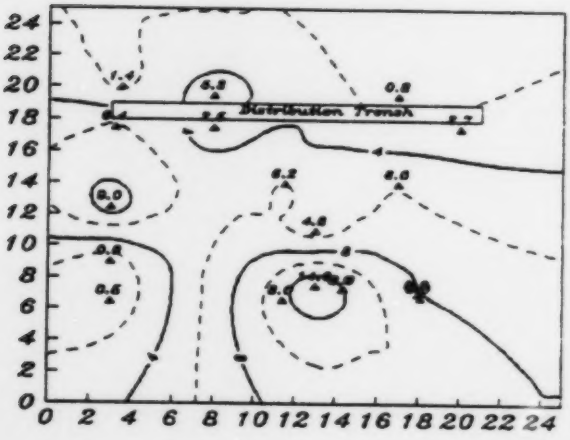
LOT 3, BOD LEVELS



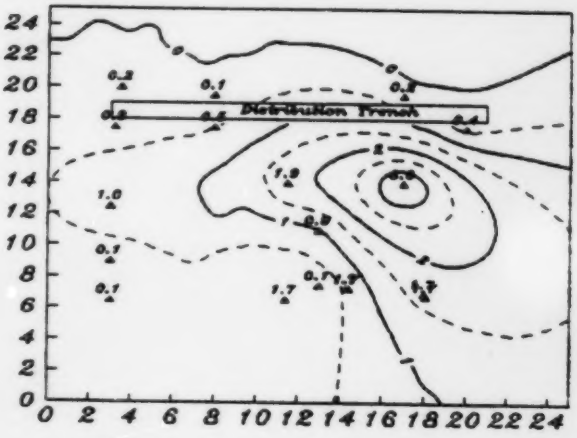
LOT 3, TOC LEVELS



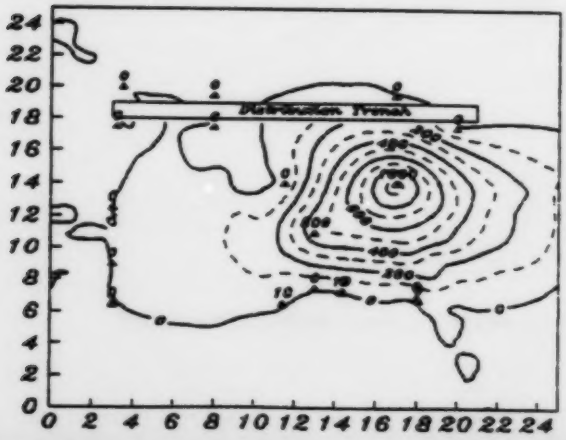
LOT 3, TKN LEVELS



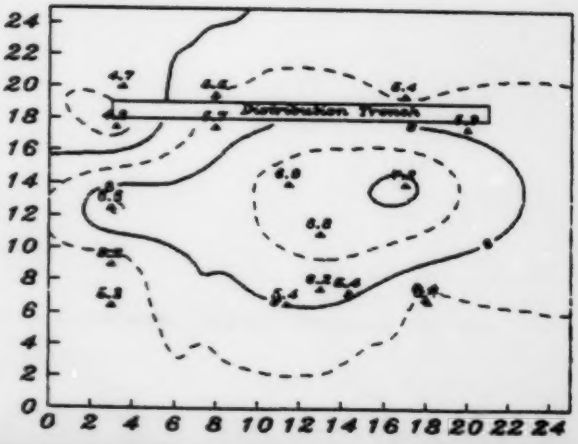
LOT 3, AMMONIA LEVELS



LOT 3, FECAL COLIFORM LEVELS



LOT 3, PH LEVELS



On-Site Related Information From New England

D.H. Waller

Contribution No. 90.-7

September 1990

Presented under the terms of Completion for the On-site Wastewater Treatment Program, 1987-90

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VISIT TO NEW ENGLAND
RE: ON-SITE SEWAGE DISPOSAL

1. Introduction

The Centre for Water Resources Studies (CWRS) undertook, in 1986, a three and one-half year research program on the subject of On-Site Sewage Disposal.

One of the projects included in that program was documentation of New England practices related to on-site systems. This project was intended to document practices and experience in other jurisdictions with similar climactic conditions, particularly in the State of Maine.

For budgetary reasons, the timing of travel to New England, and the itinerary, were dictated by opportunities presented by other travel. The information recorded here was obtained in the period May 24th to 30th of 1989.

2. Itinerary

The following visits were arranged to take advantage of a private visit by the author to the United States in May of 1989:

- May 24: P. LOMBARDO, Dames and Moore, Consulting Engineers, Boston.
- May 25: D. DEL PORTO - Water Conservation Systems Inc., Concord, Mass.
- May 26: D. HOXIE, State of Maine Dept. of Human Services, Division of Health Engineering, Augusta, Maine.
- May 30: Dr. CHET ROCK, Dept. of Civil Engineering, and Dr. GREGORY K. WHITE, Director, Environmental Studies Centre, University of Maine, Orono, Maine.

Full addresses for the individuals listed above are included in Appendix 1.

3. Dames and Moore

Pio Lombardo was called out of town and unavailable on the scheduled meeting day. The information presented here was received from him prior to the visit.

This visit was arranged because of interest in a project conducted by Mr. Lombardo in Maryland, in which a wastewater management district was organized in a region where local communities objected to installation of central services as a solution to pollution problems, because of concern about increased density of development.

The project is described in a brochure, a copy of which is attached as Appendix 2.

4. David Del Porto

Mr. Del Porto is a consultant on the subject of water conservation, and operates the firm of Water Conservation Systems, Inc.

As a consultant, he has considered effects of reduced water consumption on performance of on-site systems and on reduced energy consumption for water heating.

He has also been involved in water conservation programs intended to reduce demands on groundwater aquifers in the Boston region. These programs included plumbing code modifications to require low water-use fixtures, and a pilot program to retrofit low-flush fixtures.

Water Conservation Systems, Inc. markets a wide range of low water-use devices. A catalogue is on file with CWRS. The author purchased for his own use a 1.5 U.S. gallon American-made toilet.

5. Donald Hoxie

He provided copies of references (1) and (2).

State of Maine regulations were modified in 1974. The old regulations were based on the Manual of Septic Tank Practice, and were found impractical; percolation tests created problems in site evaluation.

Soil Conservation Service (SCS) data was used as the basis for soil-type descriptions. Table 1 (Table 16-1 from reference 2) is the basis for site evaluation. He drew attention in particular to their requirements for (a) observation holes, (b) consideration of seasonal high groundwater table and (c) soil profile analysis. In developing their regulations, they had to develop relationships between long-term acceptance rates and soil types.

The regulations recognized two situations: (1) beds with stone, and (2) leaching chambers with no stone. They believe that stones produce a shadow effect that limits the soil contact surface exposed to effluent, and increase the possibility that dirty stone and smaller particles may reduce permeability of the bio-mass. Smaller beds are therefore permitted for leaching chambers.

He thinks the regulations may require somewhat over-sized beds, but regulations are difficult to change quickly. Statistical data from inspection reports indicates no difference in failure rates in the two types of systems. Following are comments on specific points:

(1) **Site Evaluation:** Soil descriptions are easier to document, and to replicate in the future.
(2) **Water Table Effects:** Most of their soils are of till origin, with high seasonal perched water-table. He thinks systems

should be kept above the seasonal high-ground water table because there is little treatment in the saturated zone. He thinks that most treatment effects occur in the biological clogging mat.

(3) **Trenches Vs. Beds:** Trenches are allowed, but are usually too expensive.

(4) **Nitrogen:** Nitrogen removal in peat systems may be enhanced by peat as a carbon source. Several experiments have been conducted using the RUCK system for nitrogen removal. Nitrogen in groundwater is a matter of increasing concern in areas of concentrated development. A hydrogeologist in the Freeport area has developed a crude model to determine maximum lots/acre based on nitrogen loading. He uses the ratio of total nitrogen load over the water from the site reaching the permanent groundwater table. The loading is less, and construction densities higher, in clay because less water reaches the groundwater table in clay areas.

(5) **Wastewater Management Districts:** Cluster systems have been used for private condominium developments. They have no wastewater management districts, but expect that this will come. They have systems with holding tanks serving private systems.

(6) **Alternative Systems:** Several RUCK systems have been monitored, but the use is currently limited. Some mounds were installed, but were poorly constructed and unattractive, and contractors preferred level lots. Construction evolved to elevated beds in full-size systems, based on original ground. A second generation of systems includes pre-cast plastic distribution systems. They try to encourage serial distribution.

(7) **Parking Lot Systems:** Such systems are permitted, using leaching chambers (see Page 16 of Reference 1, including references to H20 loading). No evapo-transpiration from the bed is expected. No special arrangements are made to vent the system. He thinks the question is academic because he believes there is an anaerobic pond over the mat. These systems have been used in shopping centres and ski areas. I mentioned reduced water loading on the subsurface system as an effect of covering. He had not thought of this particularly, but believed it would be a benefit.

(8) **Leaching Chambers:** These systems are easier to inspect and cheaper for large installations because of their size. They are easier to install on the level, because a bed system that is level may still have beneath it an excavated area that has a low corner. Since the chambers are placed at the ground surface, it is possible to tell if the whole system is not level.

(9) **Variance Process:** This is covered in Chapter 16 of Reference 1. Credit is allowed for engineering of the system. They expect that this concept will be expanded across their regulations; this is still under development. Their points system is understandable for clients and Planning Boards and

easier for zoning administration where sensitive areas produce higher scores, and allows for unique situations within the minimum regulations. The basic concept is that you can work out a system for any lot if you spend enough money.

(10) **Water Conservation:** Design flows can be reduced if water conservation is adopted, but effects of stronger wastes resulting from conservation must be considered, i.e. the organic load is not reduced.

(11) **Materials:** Geotextiles are replacing hay, which is still widely used.

(12) **Seasonal Systems:** The requirements for cottage properties are the same as for year-round. Their legislation regarding conversions requires the ability for substantial compliance with the code, and would limit small cottage lots adjacent to lakes to seasonal use.

6. University of Maine

Dr. Chet Rock's work is reported in "Appropriate Wastewater Management Technologies for Rural Areas Under Adverse Conditions." His continuing research in this area relates to nitrogen removal in on-site system.

His experimental set-up includes a refrigerator, housing a container of septic tank effluent, from which the effluent is fed to columns via a peristaltic pump.

Other research of interest to CWRS relates to effects of overland flow through buffer strips on water quality.

He will provide reports on his research when available.

Dr. Greg White indicated that the mandate of the Environmental Studies Centre includes graduate studies, research, and outreach. Their staffing includes a person in the outreach role, supported by the Centre and State agencies. He provided copies of brochures related to: "Maine's Groundwater," "Play It Safe with Groundwater," "Access to Coastal and Inland Waters," and "The Flood of April 1987."

7. References

1. Maine Dept. of Human Services, 1987. Revisions, "Subsurface Wastewater Disposal Rules, (Chapter 241)." Division of Health Engineering, Augusta, Maine.

2. Maine Dept. of Human Services, 1987. "Site Evaluation for Subsurface Wastewater Disposal Design in Maine." Division of Health Engineering, Augusta, Maine.

APPENDIX 1

NAMES AND ADDRESSES OF THOSE VISITED IN NEW ENGLAND

Mr. Pio Lombardo - Managing Principal-in Charge
Lombardo Group
Dames and Moore
46 Church Street
Boston, Mass.
02116
(617) 451-1207

Mr. David Del Porto - Water Conservation Systems Inc.
Damonmill Square
Nine Pond Lane
Concord, Mass.
01742
(508) 369-3951

Mr. Donald Hoxie - Department of Human Services
Division of Health Engineering
State House, Station No. 10
Augusta, Maine
04333
(207) 289-3826

Dr. Chet Rock - Associate Professor of Civil Engineering
University of Maine at Orono
Orono, Maine
04469
(207) 581-2170

Dr. Gregory K. White - Director, Environmental Studies Centre
University of Maine
11 Colburn Hall
Orono, Maine
04469
(207) 581-1490

TABLE 6-1

SOIL PROFILE and CONDITION versus DISPOSAL AREA SIZE

PARENT MATERIAL	SOIL PROFILE	TEXTURAL CLASSIFICATION and DESCRIPTIONS	SOIL CONDITIONS: SHALLOW to BEDROCK A — I less than 10"	II 10" to 14"	III between [15-48"]	DRAINAGE B WELL DRAINED Ground Water Table greater than 48"	C MODERATELY WELL DRAINED Ground Water Table between [48" to 15"]	D SOMEWHAT POORLY DRAINED Ground Water Table between [15" to 6"]	E VERY POORLY DRAINED Ground Water Table less than 6"	DISPOSAL AREA RATING SEC. 12
GLACIAL TILL	1	Silt loam soils which tend to become more compact with depth. A restrictive layer may be present. Angular coarse fragments may be present.						3		LARGE
	2	Loam to sandy loam soils which do not have a restrictive layer. Angular coarse fragments may be present.						3		MEDIUM LARGE
	3	Loam, sandy loam to loamy sand soils with a restrictive layer. Angular coarse fragments may be present.						3		MEDIUM LARGE
	4	Sandy loam to loamy sand overlying loamy sand soils derived from ablation till. Coarse fragments (angular to rounded) may be present. No restrictive layer throughout profile.						3		MEDIUM
STRATIFIED DRIFT	5	Loam to sandy loam soils overlying stratified fine and medium sands. Rounded coarse fragments may be present.						3		MEDIUM
	6	Loams sand soils overlying stratified coarse sands and gravel. Round coarse fragments may be present.						3		SMALL
MIXED ORIGIN	7	Silt loam soils underlain by a restrictive layer which occurs at a depth of 15 to 24 inches. Coarse fragments may be present. Stratified lenses of sand, silt and clays usually present in lower substratum.						3		MEDIUM LARGE
LACUSTRINE MARINE	8	Loam to silt loam soils overlying firm silt loam to silty clay. A restrictive layer may be present. Coarse fragments usually absent. Stratified lenses of very fine sand, silts and clays may be present in the substratum.						3		LARGE
	9	Silt loam soils overlying firm silt loams to silty clays exhibiting a restrictive layer of coarse fragments are usually absent.						3		EXTRA LARGE
ORGANIC	10	Soils composed of organic materials in various stages of decomposition.								
ALLUVIAL, DUNE, BEACH	11	Variable in texture. Exhibiting very little weathering. Deposited in flood plain, sand dune or beach environment.				(5)	(5)			SEC. 11.F 11.G

ALL SYSTEMS PERMITTED

Note 1
See 11 C 2 a i for Separation Distances

Note 2
See 11 C 2 a ii and 11 C 2 b for Separation Distances

Severe Limitations REPLACEMENT SYSTEMS MAY BE PERMITTED (Sec. 15)

Note 3
See Sec. 15 for Replacement System Variance by LPI
See 11 C 2 a & 11 C 2 b for Separation Distances

Very Severe Limitations REPLACEMENT SYSTEMS MAY BE PERMITTED if no alternative. NEW SYSTEMS NOT PERMITTED.

Note 4
See Sec. 15 for Replacement System Variance with Department Review
See 11 C 2 for Separation Distances

Extremely Severe Limitations. SYSTEMS NOT PERMITTED

Note 5
See Sec. 11 F for Coastal Sand Dune limitations
See Sec. 11 G for Flood Plain limitations

APPENDIX 2

Mayo Peninsula
Water Reclamation Facilities

Department of Utilities
Anne Arundel County

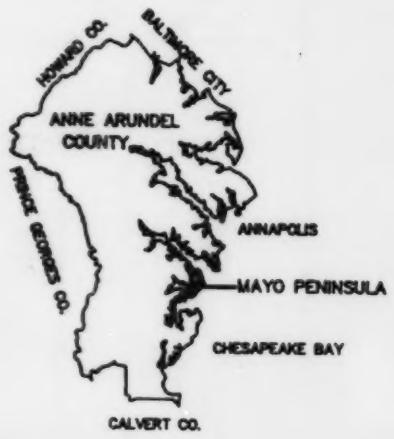
LOMBARDO & ASSOCIATES
Boston, MA Annapolis, MD

The Mayo Peninsula Water Reclamation Facilities integrate three approaches to wastewater treatment and disposal to achieve a cost effective system. These three approaches are referred to as on-site, cluster, and communal.

ON-SITE SYSTEMS — Single Home
Wherever groundwater and soil conditions allow, individual on-site septic systems will be maintained and, where needed, upgraded throughout the peninsula. More than 100 homes are proposed to be part of this system.

CLUSTER SYSTEMS — Small Groups of Homes
In two areas of the peninsula communal leaching fields will purify septic tank effluent from clusters of homes. Nearly ninety homes may be part of this system.

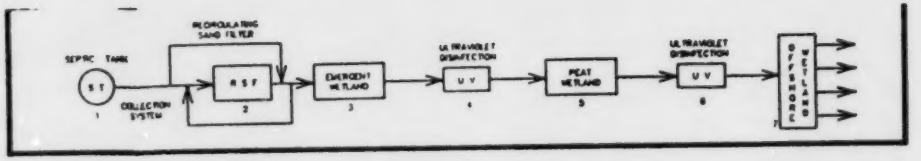
COMMUNAL TREATMENT SYSTEM — Large Community System
The remainder of the peninsula will be served by a septic tank/effluent collection system and an innovative wastewater treatment system. "Natural" wetlands, combined with other wastewater treatment techniques, work to purify the effluent to the highest quality. The steps in this treatment system are discussed on the following page.

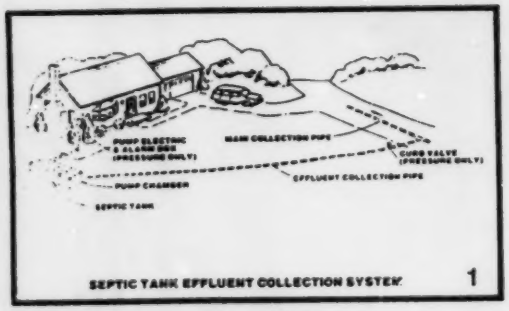


SERVICE AREA MAP

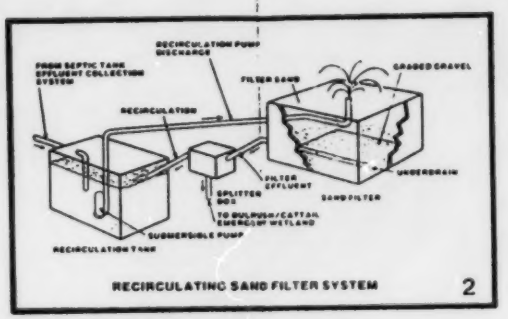
- COMMUNAL SERVICE AREA
- CLUSTER SERVICE AREA
- ON-SITE SEPTIC SYSTEM SERVICE AREA
- PARKS/OPEN SPACE

COMMUNAL TREATMENT SYSTEM PROCESS FLOW DIAGRAM

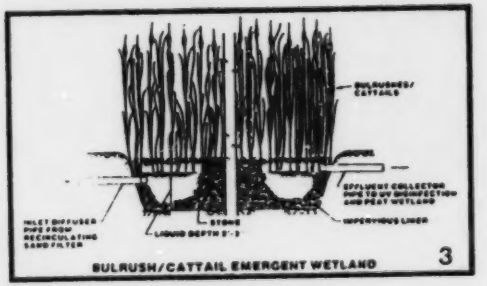




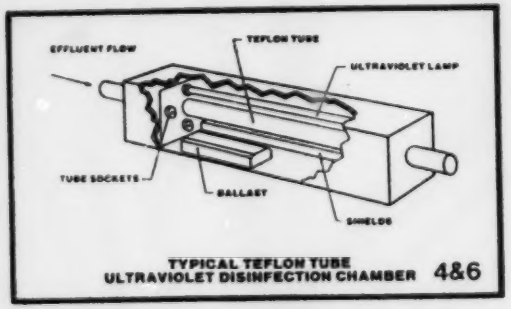
Septic Tank Effluent Pump (STEP) system. Each property will include an on-site septic tank for solids removal. Effluent flows by gravity or is pumped to a collection system and conveyed to the treatment plant.



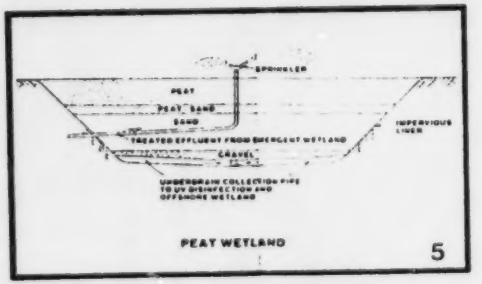
Recirculating sand filter. First, the effluent will be evenly distributed over a bed of sand, which purifies wastewater through physical and biological means



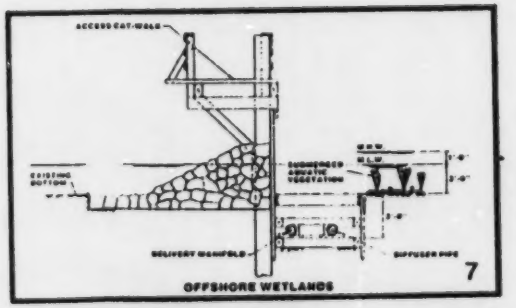
Man-made bulrush/cattail wetlands. Next, the filtered effluent will flow by gravity into man-made freshwater bulrush/cattail wetlands. Here, nitrogen will be removed and BOD, Suspended solids, and pathogens will be further reduced.



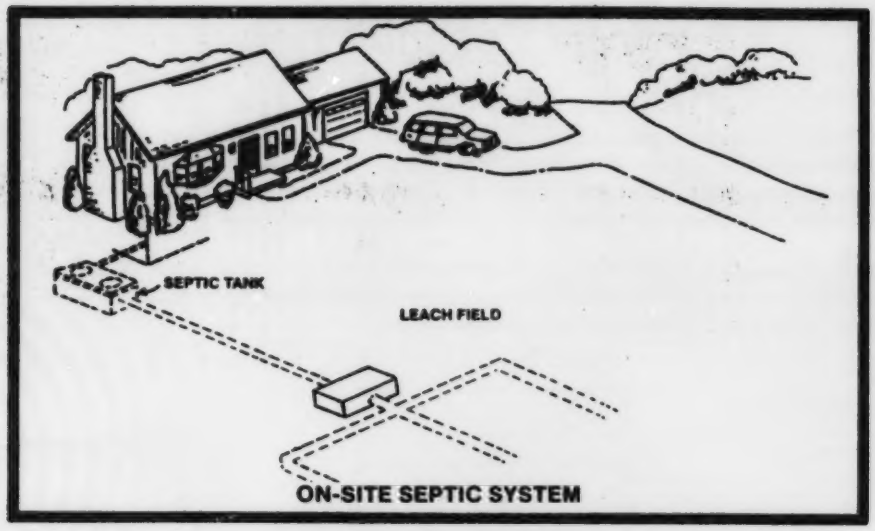
Ultraviolet disinfection. Then, the flow will pass through a chamber for ultraviolet disinfection, to eliminate pathogens.



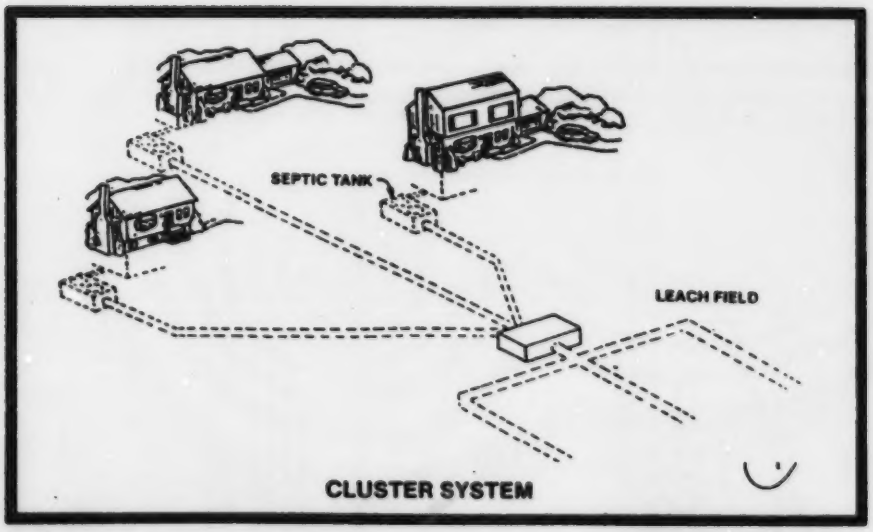
Man-made peat wetlands. For phosphorous removal, the waste water will be applied to man made peat wetlands.



Offshore wetlands. After a final ultraviolet disinfection step, the treated and disinfected effluent will be discharged into wetlands, a man-made offshore area teeming with submerged aquatic vegetation. Nearby shellfishing and water recreational activities will continue undisturbed



Individual on-site septic systems will be located where groundwater and soil conditions allow.



Septic tank effluent from clusters of homes will be purified in a communal leaching field.

PROJECT DESCRIPTION

Background. Serious wastewater and public health problems on the eight-square-mile Mayo Peninsula have plagued area residents for many years. The Anne Arundel County Department of Utilities and engineering consultants Lombardo & Associates developed a wastewater management plan to eliminate these environmental and health problems and, at the same time, address important citizen concerns, including growth management and environmental protection.

A wastewater management plan has been developed that meets with the approval of local, state, and federal environmental officials, as well as concerned residents of the peninsula. Beginning in the summer of 1986 construction on the $46-million plan will be phased in over a five-year period.

The plan is

- cost-effective
- decentralized, and
- innovative

and combines

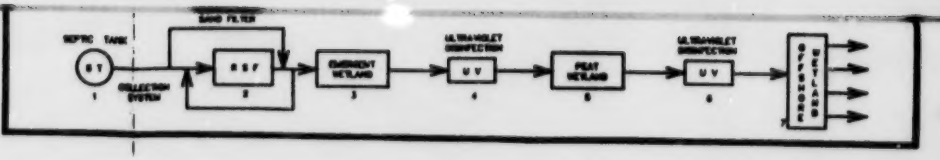
- on-site
- cluster and,
- communal treatment systems

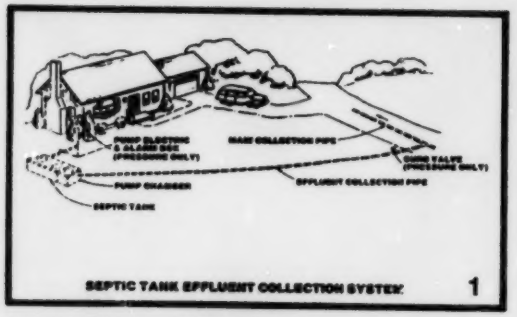
Not only does the plan solve severe wastewater problems, it enhances the natural environment through the creation of man-made inland and offshore wetlands.

Impact. This innovative plan avoids costly, growth-stimulating, centralized treatment systems. By saving $12 million, the plan demonstrates that a decentralized approach to wastewater management in a major metropolitan area can be a cost-effective option. When the entire plan is implemented, peninsula residents will again be able to enjoy the many benefits of the area without worrying about polluted wells or overflowing septic systems. This solution will actually enhance and improve upon the area's ecosystem and help restore the quality of Chesapeake Bay.

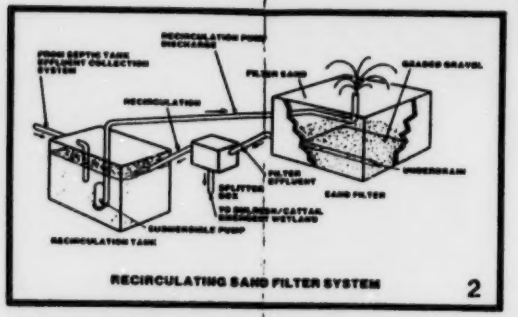
If you would like more information, the Anne Arundel County Department of Utilities will be happy to answer any questions you may have. Call or write to the Public Information Officer, Anne Arundel County Department of Utilities, 7409 Baltimore and Annapolis Boulevard, NW, Glen Burnie, MD 21061, (301) 760-7740.

COMMUNAL TREATMENT SYSTEM PROCESS FLOW DIAGRAM

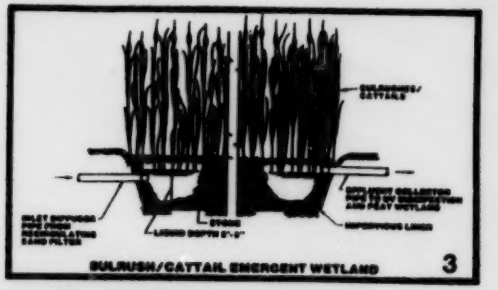




Septic Tank Effluent Pump (STEP) system. Each property will include an on-site septic tank for solids removal. Effluent flows by gravity or is pumped to a collection system and conveyed to the treatment plant.



Recirculating sand filter. First, the effluent will be evenly distributed over a bed of sand, which purifies wastewater through physical and biological means.



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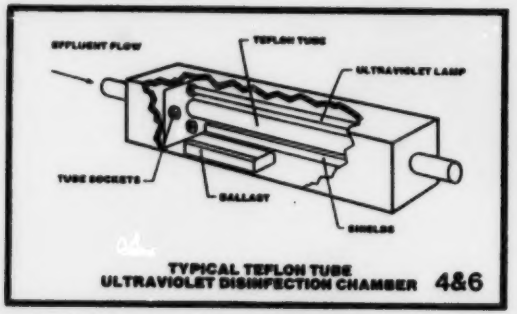
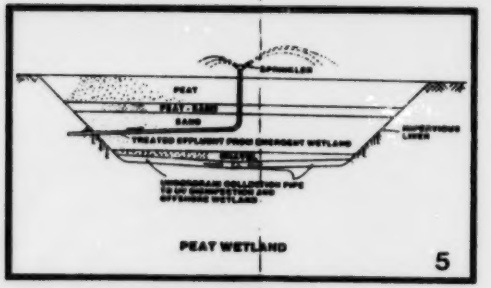
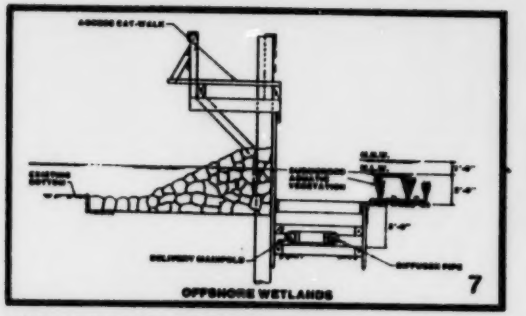


Ultraviolet disinfection. Then, the flow will pass through a chamber for ultraviolet disinfection, to eliminate pathogens.



Man-made peat wetlands. For phosphorous removal, the waste water will be applied to man-made peat wetlands.



Offshore wetlands. After a final ultraviolet disinfection step, the treated and disinfected effluent will be discharged into wetlands, a man-made offshore area teeming with submerged aquatic vegetation. Nearby shellfishing and water recreational activities will continue undisturbed.